CHAPTER 4

ANALYSIS AND DESIGN

4-1. General. Design of a pile foundation involves solving the complex problem of transferring loads from the structure through the piles to the underlying soil. It involves the analysis of a structure-pile system, the analysis of a soil-pile system, and the interaction of the two systems, which is highly nonlinear. Close cooperation between the structural engineers and geotechnical engineers is essential to the development of an effective design. This chapter addresses the criteria, procedures, and parameters necessary for the analysis and design of pile foundations.

4-2. Design Criteria.

a. Applicability and Deviations. The design criteria set forth in this paragraph are applicable to the design and analysis of a broad range of piles, soils and structures. Conditions that are site-specific may necessitate variations which must be substantiated by extensive studies and testing of both the structural properties of the piling and the geotechnical properties of the foundation.

b. Loading Conditions.

(1) Usual. These conditions include normal operating and frequent flood conditions. Basic allowable stresses and safety factors should be used for this type of loading condition.

(2) Unusual. Higher allowable stresses and lower safety factors may be used for unusual loading conditions such as maintenance, infrequent floods, barge impact, construction, or hurricanes. For these conditions allowable stresses may be increased up to 33 percent. Lower safety factors for pile capacity may be used, as described in paragraph 4-2c.

(3) Extreme. High allowable stresses and low safety factors are used for extreme loading conditions such as accidental or natural disasters that have a very remote probability of occurrence and that involve emergency maintenance conditions after such disasters. For these conditions allowable stresses may be increased up to 75 percent. Low safety factors for pile capacity may be used as described in paragraph 4-2c. An iterative (nonlinear) analysis of the pile group should be performed to determine that a state of ductile, stable equilibrium is attainable even if individual piles will be loaded to their peak, or beyond to their residual capacities. Special provisions (such as field instrumentation, frequent or continuous field monitoring of performance, engineering studies and analyses, constraints on operational or rehabilitation activities, etc.) are required to ensure that the structure will not catastrophically fail during or after extreme loading conditions. Deviations from these criteria for extreme loading conditions should be formulated in consultation with and approved by CECW-ED.

(4) Foundation Properties. Determination of foundation properties is partially dependent on types of loadings. Soil strength or stiffness, and therefore pile capacity or stiffness, may depend on whether a load is vibratory, repetitive, or static and whether it is of long or short duration.

Soil-pile properties should, therefore, be determined for each type of loading to be considered.

c. Factor of Safety for Pile Capacity. The ultimate axial capacity, based on geotechnical considerations, should be divided by the following factors of safety to determine the design pile capacity for axial loading:

Method of Determining Capacity	Loading Condition	Minimum Factor of Safety Compression	Tension
Theoretical or empirical prediction to be verified by pile load test	Usual	2.0	2.0
	Unusual	1.5	1.5
	Extreme	1.15	1.15
Theoretical or empirical prediction to be verified by pile driving analyzer as described in Paragraph 5-4a	Usual	2.5	3.0
	Unusual	1.9	2.25
	Extreme	1.4	1.7
Theoretical or empirical prediction not verified by load test	Usual	3.0	3.0
	Unusual	2.25	2.25
	Extreme	1.7	1.7

The minimum safety factors in the table above are based on experience using the methods of site investigation, testing and analysis presented herein and are the basis for standard practice. Deviations from these minimum values may be justified by extensive foundation investigations and testing which reduce uncertainties related to the variability of the foundation material and soil strength parameters to a minimum. Such extensive studies should be conducted in consultation with and approved by CECW-ED. These minimum safety factors also include uncertainties related to factors which affect pile capacity during installation and the need to provide a design capacity which exhibits very little nonlinear load-deformation behavior at normal service load levels.

d. Allowable Stresses in Structural Members. Allowable design stresses for service loads should be limited to the values described in the following paragraphs. For unusual loadings as described in paragraph 4-2b(2), the allowable stresses may be increased by one third.

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowable stress should reflect a high factor of safety. The distribution of allowable axial tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region is usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a

beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

Tension or Compression in lower pile region

Concentric axial tension or compression only 10 kips per square inch ($1/3 \times F_y \times 5/6$)	10 kips per square inch (ksi) for A-36 material
Concentric axial tension or compression only with driving shoes ($1/3 \times F_y$)	12 ksi for A-36 material
Concentric axial tension or compression only with driving shoes, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity ($1/2.5 \times F_y$)	14.5 ksi for A-36 material

Combined bending and axial compression in upper pile region:

$$\left| \frac{f_a}{F_a} \pm \frac{f_{bx}}{F_b} \pm \frac{f_{by}}{F_b} \right| \leq 1.0$$

where

f_a = computed axial unit stress

F_a = allowable axial stress

$$F_a = \frac{5}{6} \times \frac{3}{5} F_y = \frac{1}{2} F_y = 18 \text{ ksi (for A-36 material)}$$

f_{bx} and f_{by} = computed unit bending stress

F_b = allowable bending stress

$$F_b = \frac{5}{6} \times \frac{3}{5} F_y = \frac{1}{2} F_y = 18 \text{ ksi (for A-36 noncompact sections)}$$

or

$$F_b = \frac{5}{6} \times \frac{2}{3} F_y = \frac{5}{9} F_y = 20 \text{ ksi (for A-36 compact sections)}$$

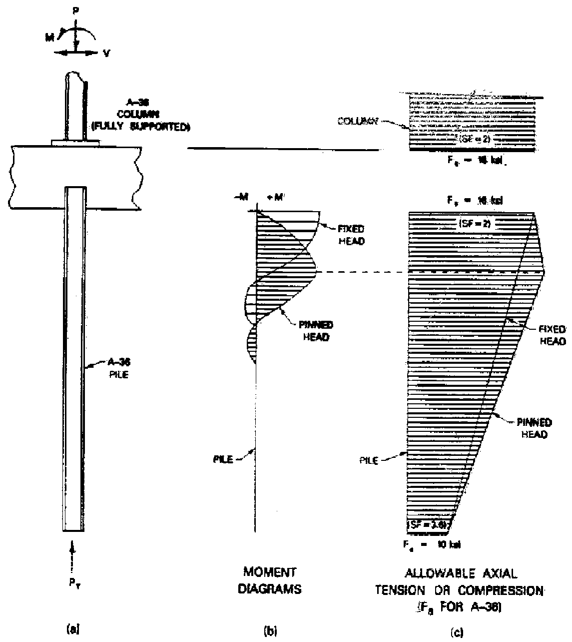

Figure 4-1. Allowable tension and compression stress for steel piles

For laterally unsupported piles the allowable stresses should be 5/6 of the American Institute of Steel Construction (AISC) (Item 21) values for beam-columns.

(2) Concrete Piles. Design criteria for four types of concrete piles (prestressed, reinforced, cast-in-place and mandrel driven) are presented in the following paragraphs.

(a) Prestressed Concrete Piles. Prestressed concrete piles are used frequently and must be designed to satisfy both strength and serviceability requirements. Strength design should follow the basic criteria set forth by the American Concrete Institute (ACI) 318 (Item 19) except the strength reduction factor (Ø) shall be 0.7 for all failure modes and the load factor shall be 1.9 for both dead and live loads. The specified load and strength reduction factors provide a safety factor equal to 2.7 for all combinations of dead and live loads. To account for accidental eccentricities, the axial strength of the pile shall be limited to 80 percent of pure axial strength, or the pile shall be designed for a minimum eccentricity equal to 10 percent of the pile width. Strength interaction diagrams for prestressed concrete piles may be developed using the computer program CPGC (Item 16). Control of cracking in prestressed piles is achieved by limiting the concrete compressive and tensile stresses under service conditions to the values indicated in Table 4-1. The allowable compressive stresses for hydraulic structures are limited to approximately 85 percent of those recommended by ACI Committee 543 (Item 20) for improved serviceability. Permissible stresses in the prestressing steel tendons should be in accordance with Item 19. A typical interaction diagram, depicting both strength and service load designs, is shown in Figure 4-2. The use of concrete with a compressive strength exceeding 7,000 psi requires

Table 4-1

Allowable Concrete Stresses, Prestressed Concrete Piles

(Considering Prestress)

Uniform Axial Tension	0
Bending (extreme fiber)	
Compression	$0.40\ f'_c$
Tension	0

For combined axial load and bending, the concrete stresses should be proportioned so that:

$$f_a + f_b + f_{pc} \leq 0.40\ f'_c$$

$$f_a - f_b + f_{pc} \geq 0$$

Where:

f_a = computed axial stress (tension is negative)

f_b = computed bending stress (tension is negative)

f_{pc} = effective prestress

f'_c = concrete compressive strength

CECW-E approval. For common uses, a minimum effective prestress of 700 psi compression is required for handling and driving purposes. Excessively long or short piles may necessitate deviation from the minimum effective prestress requirement. The capacity of piles may be reduced by slenderness effects when a portion of the pile is free standing or when the soil is too weak to provide lateral support. Slenderness effects can be approximated using moment magnification procedures. The moment magnification methods of ACI 318, as modified by PCI, "Recommended Practice for the Design of Prestressed Concrete Columns and Walls" (Item 47), are recommended.

(b) Reinforced Concrete Piles. Reinforced concrete piles shall be designed for strength in accordance with the general requirements of ACI 318 (Item 19) except as modified below. Load factors prescribed in ACI 318 should be directly applied to hydraulic structures with one alteration. The factored load combination "U" should be increased by a hydraulic load factor (H_f). This increase should lead to improved serviceability and will yield stiffer

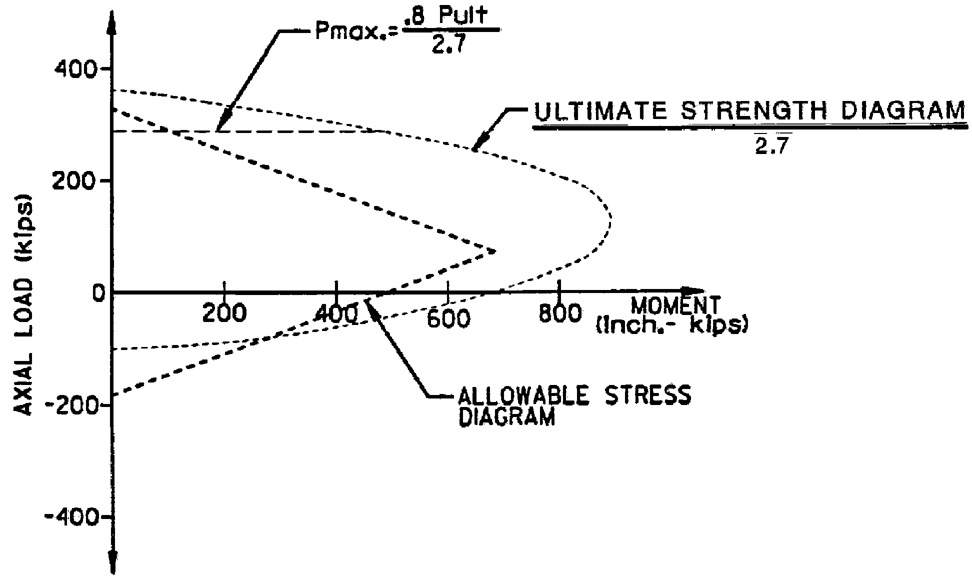

Figure 4-2. Typical interaction diagram, 16 × 16 in. square prestressed concrete pile

members than those designed solely by ACI 318. The hydraulic load factor shall be 1.3 for reinforcement calculations in flexure or compression, 1.65 for reinforcement in direct tension, and 1.3 for reinforcement in diagonal tension (shear). The shear reinforcement calculation should deduct the shear carried by the concrete prior to application of the hydraulic load factor. As an alternate to the prescribed ACI load factors, a single load factor of 1.7 can be used. The 1.7 should then be multiplied by H_f. The axial compression strength of the pile shall be limited to 80 percent of the ultimate axial strength, or the pile shall be designed for a minimum eccentricity equal to 10 percent of the pile width. Strength interaction diagrams for reinforced concrete piles may be developed using the Corps computer program CASTR (Item 18). Slenderness effects can be approximated using the ACI moment magnification procedures.

(c) Cast-in-Place and Mandrel-Driven Piles. For a cast-in-place pile, the casing is top-driven without the aid of a mandrel, and the casing typically has a wall thickness ranging from 9 gage to 1/4 inch. The casing must be of sufficient thickness to withstand stresses due to the driving operation and maintain the cross section of the pile. The casing thickness for mandrel-driven piles is normally 14 gage. Cast-in-place and mandrel-driven piles should be designed for service conditions and stresses limited to those values listed in Table 4-2. The allowable compressive stresses are reduced from those recommended by ACI 543 (Item 20), as explained for prestressed concrete piles. Cast-in-place and mandrel-driven piles shall be used only when full embedment and full lateral support are assured and under conditions which produce zero or small end moments, so that compression always controls. In order for a pile to qualify as confined, the steel casing must be 14 gage (US Standard) or thicker, be seamless or have spirally welded seams, have a minimum yield strength of 30 ksi, be 17 inches or less in diameter, not be exposed to a detrimental corrosive environment, and not be designed to carry a

Table 4-2

Cast-in-Place and Mandrel-Driven Piles, Allowable Concrete Stresses

(Participation of steel casing or shell disallowed)

Uniform Axial Compression	
Confined	$0.33\ f'_c$
Unconfined	$0.27\ f'_c$
Uniform Axial Tension	0
Bending (extreme fiber)	
Compression	$0.40\ f'_c$
Tension	0

For combined axial load and bending, the concrete stresses should be proportioned so that:

$$\left| \frac{f_a}{F_a} \pm \frac{f_b}{F_b} \right| \leq 1.0$$

Where:

f_a = computed axial stress

F_a = allowable axial stress

f_b = computed bending stress

F_b = allowable bending stress

portion of the working load. Items not specifically addressed in this paragraph shall be in accordance with ACI 543.

(3) Timber Piles. Representative allowable stresses for pressure-treated round timber piles for normal load duration in hydraulic structures are:

Species	Compression Parallel to Grain (psi) F_a	Bending (psi) F_b	Horizontal Shear (psi)	Compression Perpendicular to Grain (psi)	Modulus of Elasticity (psi)
Pacific Coast (a)* Douglas Fir	875	1,700	95	190	1,500,000
Southern Pine (a)(b)*	825	1,650	90	205	1,500,000

(a) The working stresses for compression parallel to grain in Douglas Fir and Southern Pine may be increased by 0.2 percent for each foot of length from the tip of the pile to the critical section. For compression perpendicular to grain, an increase of 2.5 psi per foot of length is recommended.

(b) Values for Southern Pine are weighted for longleaf, slash, loblolly and shortleaf representatives of piles in use.

(c) The above working stresses have been adjusted to compensate for strength reductions due to conditioning and treatment. For untreated piles or piles that are air-dried or kiln-dried before pressure treatment, the above working stresses should be increased by dividing the tabulated values by the following factors:

Pacific Coast Douglas Fir:	0.90
Southern Pine:	0.85

(d) The allowable stresses for compression parallel to the grain and bending, derived in accordance with ASTM D2899, are reduced by a safety factor of 1.2 in order to comply with the general intent of Paragraph 13.1 of ASTM D2899 (Item 22).

(e) For hydraulic structures, the above values, except for the modulus of elasticity, have been reduced by dividing by a factor of 1.2. This additional reduction recognizes the difference in loading effects between the ASTM normal load duration and the longer load duration typical of hydraulic structures, and the uncertainties regarding strength reduction due to conditioning processes prior to treatment. For combined axial load and bending, stresses should be so proportioned that:

$$\left| \frac{f_a}{F_a} \pm \frac{f_b}{F_b} \right| \leq 1.0$$

where

f_a = computed axial stress

F_a = allowable axial stress

f_b = computed bending stress

F_b = allowable bending stress

e. Deformations. Horizontal and vertical displacements resulting from applied loads should be limited to ensure proper operation and integrity of the structure. Experience has shown that a vertical deformation of 1/4 inch and a lateral deformation of 1/4 to 1/2 inch at the pile cap are representative of long-term movements of structures such as locks and dams. Operational requirements may dictate more rigid restrictions and deformations. For other structures such as piers, larger deformations may be allowed if the stresses in the structure and the piles are not excessive. Since the elastic spring constants used in the pile group analysis discussed later are based on a linear load versus deformation relationship at a specified deformation, it is important to keep the computed deformations at or below the specified value. Long-term lateral deformations may be larger than the computed values or the values obtained from load tests due to creep or plastic flow. Lateral deflection may also increase due to cyclic loading and close spacing. These conditions should be investigated when determining the maximum predicted displacement.

f. Allowable Driving Stresses. Axial driving stresses calculated by wave equation analysis should be limited to the values shown in Figure 4-3.

g. Geometric Constraints.

(1) Pile Spacing. In determining the spacing of piles, consideration should be given to the characteristics of the soil and to the length, size, driving tolerance, batter, and shape of the piles. If piles are spaced too closely, the bearing value and lateral resistance of each pile will be reduced, and there is danger of heaving of the foundation, and uplifting or damaging other piles already driven. In general, it is recommended that end-bearing piles be spaced not less than three pile diameters on centers and that friction piles, depending on the characteristics of the piles and soil, be spaced a minimum of three to five pile diameters on center. Piles must be spaced to avoid tip interference due to specified driving tolerances. See paragraph 5-2a(3) for typical tolerances. Pile layouts should be checked for pile interference using CPGI, a program which is being currently developed and is discussed in paragraph 1-3c(b).

(2) Pile Batter. Batter piles are used to support structures subjected to large lateral loads, or if the upper foundation stratum will not adequately resist lateral movement of vertical piles. Piles may be battered in opposite directions or used in combination with vertical piles. The axial load on a batter pile should not exceed the allowable design load for a vertical pile. It is very difficult to drive piles with a batter greater than 1 horizontal to 2 vertical. The driving efficiency of the hammer is decreased as the batter increases.

Extended Driving and Seating Using WEAP

Pile Material	Allowable Driving Stress
Steel	$0.85\ F_y$
Reinforced Concrete	
Compression	$0.85\ f'_c$
Tension	500 psi
Prestressed Concrete	
Compression	$(0.85\ f'_c - f_{pe})$
Tension	f_{pe}
Timber	3000 psi

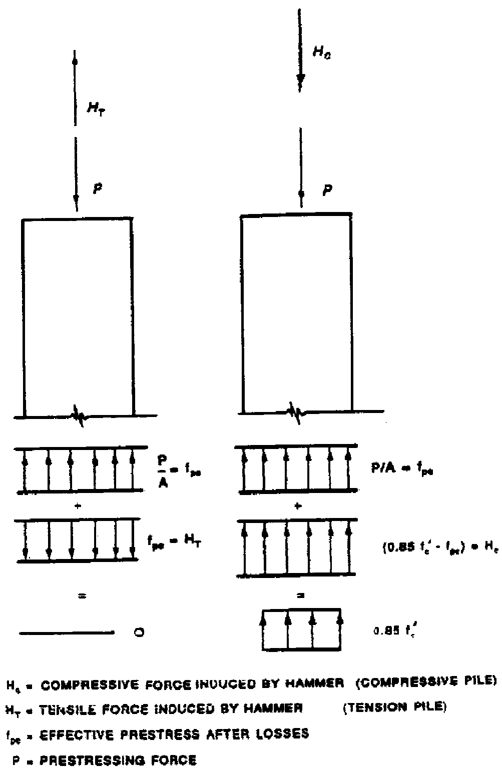

Figure 4-3. Prestressed concrete pile driving stresses

4-3. Pile Capacity. Pile capacities should be computed by experienced designers thoroughly familiar with the various types of piles, how piles behave when loaded, and the soil conditions that exist at the site.

a. Axial Pile Capacity. The axial capacity of a pile may be represented by the following formula:

$$Q_{ult} = Q_s + Q_t$$

$$Q_s = f_s A_s$$

$$Q_t = q A_t$$

where

Q_{ult} = ultimate pile capacity

Q_s = shaft resistance of the pile due to skin friction

Q_t = tip resistance of the pile due to end bearing

f_s = average unit skin resistance

A_s = surface area of the shaft in contact with the soil

q = unit tip-bearing capacity

A_t = effective (gross) area of the tip of the pile in contact with the soil

(1) Piles in Cohesionless Soil.

(a) Skin Friction. For design purposes the skin friction of piles in sand increase linearly to an assumed critical depth (D_c) and then remain constant below that depth. The critical depth varies between 10 to 20 pile diameters or widths (B), depending on the relative density of the sand. The critical depth is assumed as:

$D_c = 10B$ for loose sands

$D_c = 15B$ for medium dense sands

$D_c = 20B$ for dense sands

The unit skin friction acting on the pile shaft may be determined by the following equations:

$$f_s = K\sigma_v' \tan\delta$$

$$\sigma_v' = \gamma' D \quad \text{for} \quad D < D_c$$

$$\sigma_v' = \gamma' D_c \quad \text{for} \quad D \geq D_c$$

$$Q_s = f_s A_s$$

where

K = lateral earth pressure coefficient (K_c for compression piles and K_t for tension piles)

σ_v' = effective overburden pressure

δ = angle of friction between the soil and the pile

γ' = effective unit weight of soil

D = depth along the pile at which the effective overburden pressure is calculated

Values of δ are given in Table 4-3.

Table 4-3

Values of δ

Pile Material	δ
Steel	0.67 ϕ to 0.83 ϕ
Concrete	0.90 ϕ to 1.0 ϕ
Timber	0.80 ϕ to 1.0 ϕ

Values of K for piles in compression (K_c) and piles in tension (K_t) are given in Table 4-4. Table 4-3 and Table 4-4 present ranges of values of δ and K based upon experience in various soil deposits. These values should be selected for design based upon experience and pile load test. It is not intended that the designer would use the minimum reduction of the ϕ angle while using the upper range K values.

Table 4-4

Values of K

Soil Type	K_c	K_t
Sand	1.00 to 2.00	0.50 to 0.70
Silt	1.00	0.50 to 0.70
Clay	1.00	0.70 to 1.00

Note: The above do not apply to piles that are prebored, jetted, or installed with a vibratory hammer. Picking K values at the upper end of the above ranges should be based on local experience. K , δ , and N_q values back calculated from load tests may be used.

For steel H-piles, A_s should be taken as the block perimeter of the pile and δ should be the average friction angles of steel against sand and sand against sand (ϕ). It should be noted that Table 4-4 is general guidance to be used unless the long-term engineering practice in the area indicates otherwise. Under prediction of soil strength parameters at load test sites has at times produced back-calculated values of K that exceed the values in Table 4-4. It has also been found both theoretically and at some test sites that the use of displacement piles produces higher values of K than does the use of nondisplacement piles. Values of K that have been used satisfactorily but with standard soil data in some locations are as follows in Table 4-5:

Table 4-5

Common Values for Corrected K

	Displacement Piles		Nondisplacement Piles	
Soil Type	Compression	Tension	Compression	Tension
Sand	2.00	0.67	1.50	0.50
Silt	1.25	0.50	1.00	0.35
Clay	1.25	0.90	1.00	0.70

Note: Although these values may be commonly used in some areas they should not be used without experience and testing to validate them.

(b) End Bearing. For design purposes the pile-tip bearing capacity can be assumed to increase linearly to a critical depth (D_c) and then remains constant. The same critical depth relationship used for skin friction can be used for end bearing. The unit tip bearing capacity can be determined as follows:

$$q = \sigma_v' N_q$$

where:

$$\sigma_v' = \gamma' D \quad \text{for} \quad D < D_c$$

$$\sigma_v' = \gamma' D_c \quad \text{for} \quad D \geq D_c$$

For steel H-piles A_t should be taken as the area included within the block perimeter. A curve to obtain the Terzaghi-Peck (Item 59) bearing capacity factor N_q (among values from other theories) is shown in Figure 4-4. To use the curve one must obtain measured values of the angle of internal friction (ϕ) which represents the soil mass.

(c) Tension Capacity. The tension capacity of piles in sand can be calculated as follows using the K values for tension from Table 4-4:

$$Q_{ult} = Q_{s_{tension}}$$

(2) Piles in Cohesive Soil.

(a) Skin Friction. Although called skin friction, the resistance is due to the cohesion or adhesion of the clay to the pile shaft.

$$f_s = c_a$$

$$c_a = \alpha c$$

$$Q_s = f_s A_s$$

where

c_a = adhesion between the clay and the pile

α = adhesion factor

c = undrained shear strength of the clay from a Q test

The values of α as a function of the undrained shear are given in Figure 4-5a.

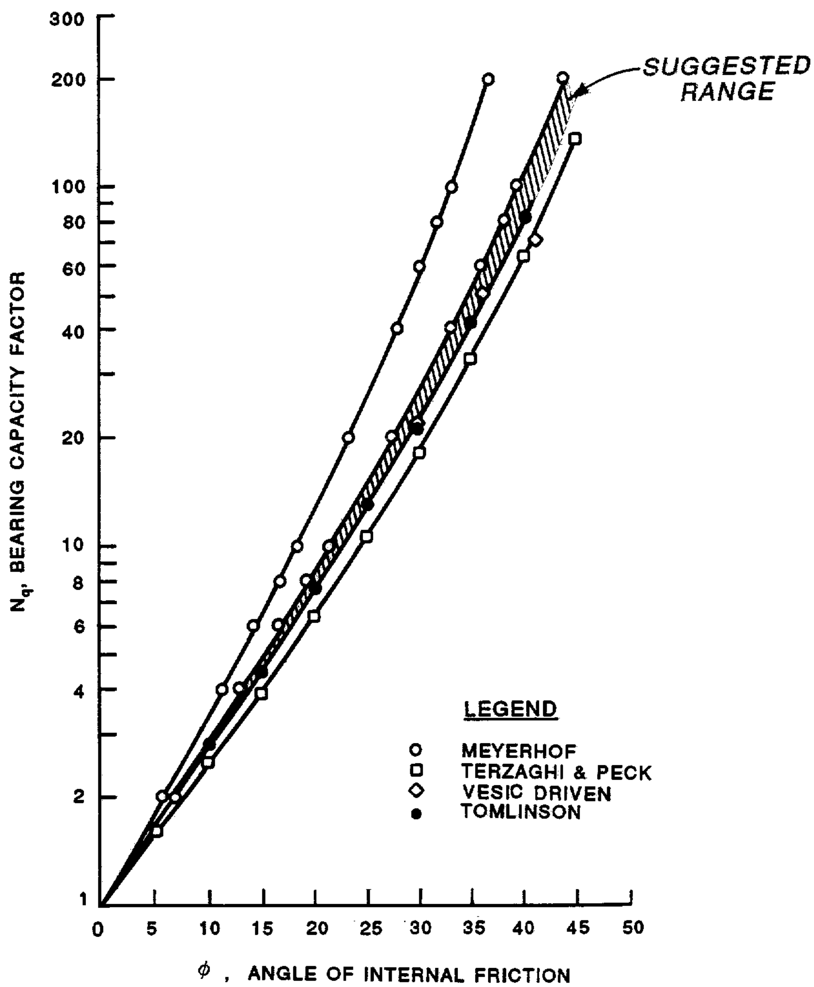

Figure 4-4. Bearing capacity factor

An alternate procedure developed by Semple and Rigden (Item 56) to obtain values of α which is especially applicable for very long piles is given in Figure 4-5b where:

$$\alpha = \alpha_1 \alpha_2$$

and

$$f_s = \alpha c$$

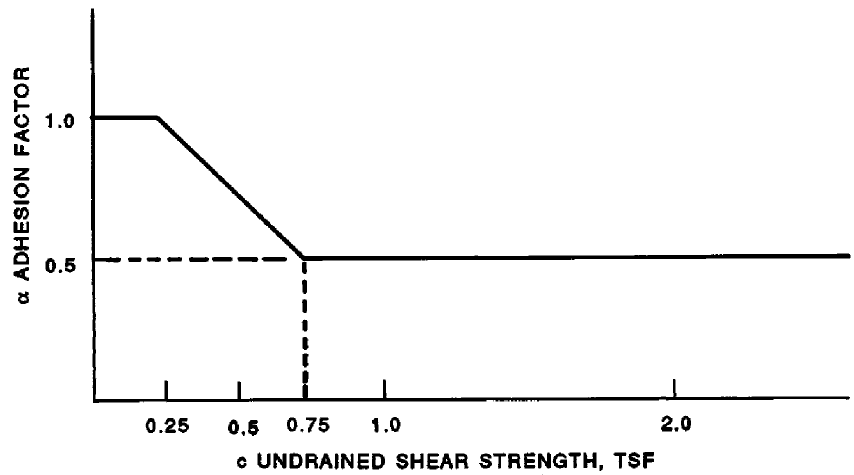

Figure 4-5a. Values of α versus undrained shear strength

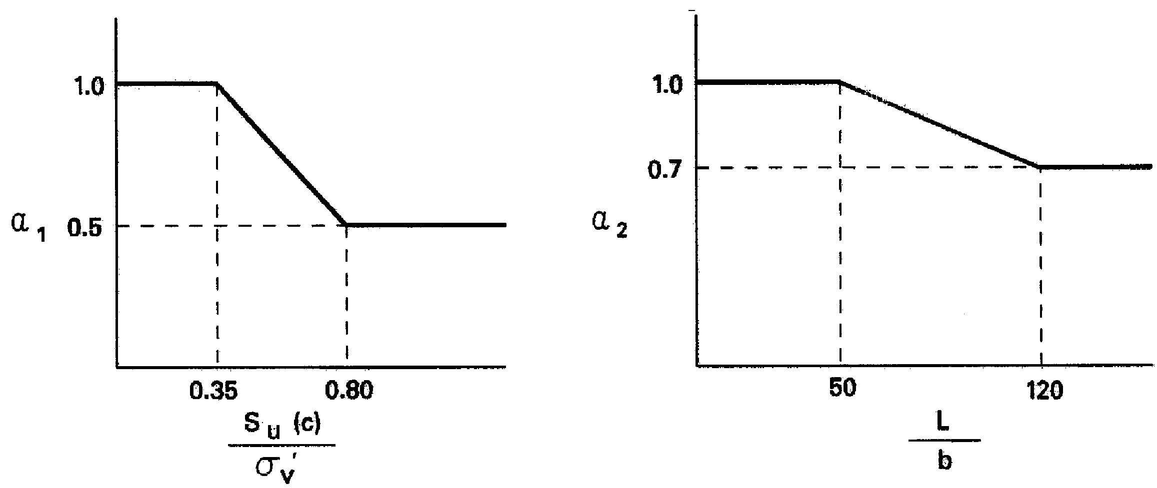

Figure 4-5b. Values of α_1 α_2 applicable for very long piles

(b) End Bearing. The pile unit-tip bearing capacity for piles in clay can be determined from the following equation:

$$q = 9c$$

$$Q_t = A_t q$$

However, the movement necessary to develop the tip resistance of piles in clay soils may be several times larger than that required to develop the skin friction resistance.

(c) Compression Capacity. By combining the skin friction capacity and the tip bearing capacity, the ultimate compression capacity may be found as follows:

$$Q_{ult} = Q_s + Q_t$$

(d) Tension Capacity. The tension capacity of piles in clay may be calculated as:

$$Q_{ult} = Q_s$$

(e) The pile capacity in normally consolidated clays (cohesive soils) should also be computed in the long-term S shear strength case. That is, develop a S case shear strength trend as discussed previously and proceed as if the soil is drained. The computational method is identical to that presented for piles in granular soils, and to present the computational methodology would be redundant. It should be noted however that the shear strengths in clays in the S case are assumed to be $\phi > 0$ and $C = 0$. Some commonly used S case shear strengths in alluvial soils are as follows in Table 4-6:

Table 4-6

S Case Shear Strength

Soil Type	Consistency	Angle of Internal Friction ϕ
Fat clay (CH)	Very soft	13° to 17°
Fat clay (CH)	Soft	17° to 20°
Fat clay (CH)	Medium	20° to 21°
Fat clay (CH)	Stiff	21° to 23°
Silt (ML)		25° to 28°

Note: The designer should perform testing and select shear strengths. These general data ranges are from test on specific soils in site specific environments and may not represent the soil in question.

(3) Piles in Silt.

(a) Skin Friction. The skin friction on a pile in silt is a two component resistance to pile movement contributed by the angle of internal friction (ϕ) and the cohesion (c) acting along the pile shaft. That portion of the resistance contributed by the angle of internal friction (ϕ) is as with the sand limited to a critical depth of (D_c), below which the frictional portion remains constant, the limit depths are stated below. That portion of the resistance contributed by the cohesion may require limit if it is sufficiently large, see Figures 4-5a and b. The shaft resistance may be computed as follows:

$$K\gamma' D \tan \delta + \alpha c$$

$$\text{where } (D \le D_c)$$

$$Q_s = A_s f_s$$

where

Q_s = capacity due to skin resistance

f_s = average unit skin resistance

A_s = surface area of the pile shaft in contact with soil

K = see Table 4-4

α = see Figures 4-5a and b

D = depth below ground up to limit depth D_c

δ = limit value for shaft friction angle from Table 4-3

(b) End Bearing. The pile tip bearing capacity increases linearly to a critical depth (D_c) and remains constant below that depth. The critical depths are given as follows:

$$D_c = 10\ B \text{ for loose silts}$$

$$D_c = 15\ B \text{ for medium silts}$$

$$D_c = 20\ B \text{ for dense silts}$$

The unit and bearing capacity may be computed as follows:

$$q = \sigma'_v N_q$$

$$\sigma'_v = \gamma' D \quad \text{for} \quad D < D_c$$

$$\sigma'_v = \gamma' D_c \quad \text{for} \quad D \geq D_c$$

$$Q_t = A_t q$$

where

N_q = Terzaghi bearing capacity factor, Figure 4-4

σ'_v = vertical earth pressure at the tip with limits

A_t = area of the pile tip, as determined for sands

(c) Compression Capacity. By combining the two incremental contributors, skin friction and end bearing the ultimate capacity of the soil/pile may be computed as follows:

$$Q_{ult} = Q_s + Q_t$$

(d) Tension Capacity. The tension capacity is computed by applying the appropriate value of K_t from Table 4-4 to the unit skin friction equation above.

$$Q_{ult} = Q_{s_{tension}}$$

(e) It is recommended that when designing pile foundations in silty soils, considerations be given to selecting a very conservative shear strength from classical R shear tests. It is further recommended that test piles be considered as a virtual necessity, and the possibility that pile length may have to be increased in the field should be considered.

(4) Piles in Layered Soils. Piles are most frequently driven into a layered soil stratigraphy. For this condition, the preceding methods of computation may be used on a layer by layer basis. The end bearing capacity of the pile should be determined from the properties of the layer of soil where the tip is founded. However, when weak or dissimilar layers of soil exist within approximately 5 feet or 8 pile tip diameters, whichever is the larger, of the tip founding elevation the end bearing capacity will be affected. It is necessary to compute this affect and account for it when assigning end bearing capacity. In computing the skin resistance, the contribution of each layer is computed separately, considering the layers above as a surcharge and applying the appropriate reduction factors for the soil type within that increment of pile shaft.

(a) Skin Friction. The skin friction contributed by different soil types may be computed incrementally and summed to find the ultimate capacity. Consideration should be given to compatibility of strain between layers when computing the unit skin resistance.

$$Q_s = \sum_{i=1}^{N} f_{s_i} A_{s_i}$$

where

f_{s_i} = unit skin resistance in layer i

A_{s_i} = surface area of pile in contact with layer i

N = total number of layers

(b) End Bearing. The pile tip bearing should be computed based upon the soil type within which the tip is founded, with limits near layer boundaries mentioned above. Using the overlying soil layers as surcharge the following equations may be used.

Sand or Silt: $q = \sigma'_v N_q$

$$\sigma'_v = \gamma' D \quad \text{for} \quad D < D_c$$

$$\sigma'_v = \gamma' D_c \quad \text{for} \quad D > D_c$$

$$Q_t = A_t q$$

Clay: $q = 9c$

$$Q_t = A_t q$$

(c) Compression Capacity. By combining the skin resistance and end bearing, the ultimate capacity of the soil/pile may be computed as follows:

$$Q_{ult} = Q_s + Q_t$$

(d) Tension Capacity. The tension capacity may be computed by applying the appropriate values of K_t from Table 4-4 as appropriate for granular soils to the incremental computation for each layer and then combining to yield:

$$Q_{ult} = Q_{s_{tension}}$$

(5) Point Bearing Piles. In some cases the pile will be driven to refusal upon firm good quality rock. In such cases the capacity of the pile is governed by the structural capacity of the pile or the rock capacity.

(6) Negative Skin Friction.

(a) Negative skin friction is a downward shear drag acting on piles due to downward movement of surrounding soil strata relative to the piles. For such movement of the soils to occur, a segment of the pile must penetrate a compressible soil stratum that consolidates. The downward drag may be caused by the placement of fill on compressible soils, lowering of the groundwater table, or underconsolidated natural or compacted soils. The effect of these occurrences is to cause the compressible soils surrounding the piles to consolidate. If the pile tip is in a relatively stiff soil, the upper compressible stratum will move down relative to the pile, inducing a drag load. This load can be quite large and must be added to the structural load for purposes of assessing stresses in the pile. Vesic (Item 60) stated that a relative downward movement of as little as 0.6 inch of the soil with respect to the pile may be sufficient to mobilize full negative skin friction. The geotechnical capacity of the pile is unaffected by downdrag, however downdrag does serve to increase settlement and increase the stresses in the pile and pile cap.

(b) For a pile group, it can be assumed that there is no relative movement between the piles and the soil between the piles. Therefore, the total force acting down is equal to the weight of the block of soil held between the piles, plus the shear along the pile group perimeter due to negative skin friction. The average downward load transferred to a pile in a pile group Q_{nf} can be estimated by

$$Q_{nf} = \frac{1}{N} [A\gamma L + sLP] \tag{1}$$

where

A = horizontal area bounded by the pile group (cross-sectional area of piles and enclosed soil)

N = number of piles in pile group

γ = unit weight of fill or compressible soil layers

L = length of embedment above the bottom of the compressible soil layers

s = shear resistance of the soil

P = perimeter of the area A

(c) For a single pile, the downward load transferred to the pile is equal to the shearing resistance along the pile as shown in Equation 2.

$$Q_{nf} = sLP' \tag{2}$$

where P' = perimeter of pile. The total applied load (Q_T) on a pile group or single pile is the live load, dead load, and the drag load due to negative skin friction.

$$Q_T = Q + A\gamma L + sLP \text{ (pile group)} \tag{3a}$$

$$Q_T = Q + sLP' \qquad \text{(single pile)} \tag{3b}$$

where Q = live load plus dead load.

(d) Commentary. Equation 1 for pile groups was used by Teng (Item 58) and Terzaghi and Peck (Item 59). However, in Peck, Hanson, and Thornburn (Item 46), the shear resistance on the perimeter was eliminated. Both Teng and Terzaghi and Peck state that the component due to shear resistance is the larger value. Teng recommends using the lesser of the summation of shear resistance for individual piles of a pile group and Equation 1. Bowles (Item 27) and the Department of the Navy, Naval Facilities Engineering Command (NAVFAC) (Item 33) both use a coefficient relating the overburden pressures to the shearing resistance around the pile. NAVFAC gives different values for clay, silt, and sands and references Garlanger (Item 35), Prediction of Downdrag Load at the Cutler Circle Bridge. Bowles uses the block perimeter resistance for a pile group similar to Equation 1. Bowles recommends using the higher value of Equation 1 and, between the summation of shear resistance on a single pile, using the coefficient relating overburden pressure to shear resistance and Equation 1. NAVFAC does not use the block perimeter resistance for a pile group. For single piles, NAVFAC uses the coefficient times the effective vertical stress.

b. Pile Group Capacity. The pile group capacity for piles in cohesionless and cohesive soils is given below.

(1) Piles in Cohesionless Soil. The pile group efficiency η is defined as:

$$\eta = \frac{Q_{group}}{NQ_{ult}}$$

where

Q_{group} = ultimate capacity of a pile group

N = number of piles in a group

Q_{ult} = ultimate capacity of a single pile

The ultimate group capacity of driven piles in sand is equal to or greater than the sum of the ultimate capacity of the single piles. Therefore in practice, the ultimate group capacity of driven piles in sand not underlain by a weak layer, should be taken as the sum of the single pile capacities ($\eta = 1$). For piles jetted into sand, η is less than one. For piles underlain by a weak layer, the ultimate group capacity is the smaller of (a) the sum of the single pile ultimate capacities or (b) the capacity of an equivalent pier with the geometry defined by enclosing the pile group (Item 59). The base strength should be that of the weak layer.

(2) Piles in Cohesive Soil. The ultimate group capacity of piles in clay is the smaller of (a) the sum of the single pile ultimate capacities or (b) the capacity of an equivalent pier (Item 59). The ultimate group capacity of piles in clay is given by the smaller of the following two equations:

$$Q_{group} = NQ_{ult}$$

$$Q_{group} = 2(\mathbf{B}_g + L_g)D\bar{c} + \left[5 \left(1 + \frac{D}{5\mathbf{B}_g}\right) \left(1 + \frac{\mathbf{B}_g}{5L_g}\right) \right] c_b L_G \mathbf{B}_G$$

where

$$N_c = 5\left(1 + \frac{D}{5B_g}\right)\left(1 + \frac{B_g}{5L_g}\right) \leq 9$$

and:

B_g = width of the pile group

L_g = length of the pile group

D = depth of the pile group

$\bar{c}$ = weighted average of undrained shear strength over the depth of pile embedment. $\bar{c}$ should be reduced by α from Figure 4-5.

c_b = undrained shear strength at the base of the pile group

This equation applies to a rectangular section only. It should be modified for other shapes.

4-4. Settlement. The load transfer settlement relationship for single piles and pile groups is very complex. Most settlement analysis methods are based on empirical methods and give only a rough approximation of the actual settlement. However, settlements of single piles and pile groups should be calculated to give the designer a perception of how the structure will perform and to check that these calculated settlements are within acceptable limits (paragraph 4-2e). Calculated foundation settlements should be compatible with the force-movement relationships used in designing the structure.

a. Single Piles.

(1) Semi-Empirical Method. The semi-empirical method for calculating the settlement of single piles is the method proposed by Vesic (Item 60). The settlement of a single pile is given by the equation

$$w = w_s + w_{pp} + w_{ps}$$

where

w = vertical settlement of the top of a single pile

w_s = amount of settlement due to the axial deformation of the pile shaft

w_{pp} = amount of settlement of the pile tip due to the load transferred at the tip

w_{ps} = amount of settlement of the pile tip caused by load transmitted along the pile shaft

The axial deformation of the pile shaft is given by:

$$w_s = (Q_p + \alpha_s Q_s)\frac{L}{AE}$$

where

Q_p = tip resistance of the pile for the design load for which the settlement is being calculated

α_s = number that depends on the skin friction distribution along the pile (Figure 4-6)

Q_s = shaft resistance of the pile for the design load for which the settlement is being calculated

L = length of the pile

A = cross-sectional area of the pile

E = modulus of elasticity of the pile

Lesser values of α_s have been observed in long driven piles subject to hard driving. A typical value for piles driven into dense sand may be around 0.1. Lesser values of α_s are also observed for long, flexible friction piles where under working loads, only a fraction of the shaft length transmits load. The settlement at the tip of the pile can be calculated by the following equations:

$$w_{pp} = \frac{C_p Q_p}{Bq}$$

$$w_{ps} = \frac{C_s Q_s}{Dq}$$

where

C_p = empirical coefficient given in Table 4-7

B = pile diameter or width

q = unit ultimate tip bearing capacity

C_s = coefficient given by the following equation
$C_s = (0.93 + 0.16 \sqrt{D/B}\,) C_p$

D = embedded pile length

The values of C_p given in Table 4-7 are for long-term settlement of the pile where the bearing stratum beneath the pile tip extends a minimum of 10B beneath the pile tip and where such soil is of equal or higher stiffness than that of the soil at the tip elevation. The value of C_p will be lower if rock exists nearer the pile tip than 10B. If rock exists at 5B beneath the pile tip, use 88 percent of w_{pp} in the settlement calculations. If rock

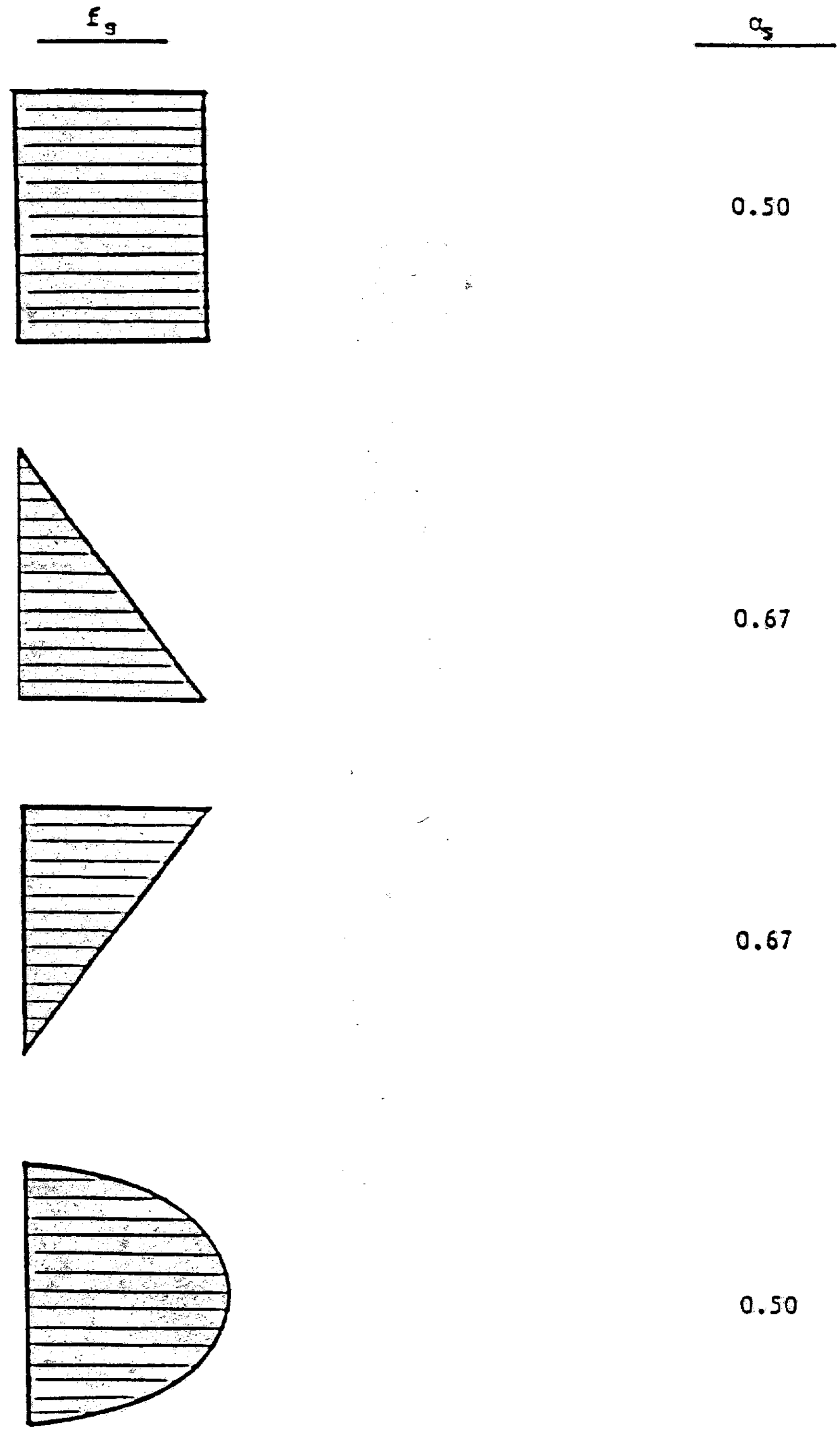

Figure 4-6. Values of α_s for different skin friction distributions (Item 60)

exists at 1B beneath the pile tip, use 51 percent of w_{pp} in the settlement calculations. Unless a highly compressible layer exists beneath the pile tip, consolidation settlement should not be significant and normally does not exceed 15 percent of the total settlement. If a highly compressible layer does exist beneath the pile tip, a consolidation-settlement analysis should be performed to determine the additional long-term settlement that will occur.

Table 4-7

Value of C_p

Soil Type	Driven Piles	Bored Piles
Sand (dense to loose)	0.02 to 0.04	0.09 to 0.18
Clay (stiff to soft)	0.02 to 0.03	0.03 to 0.06
Silt (dense to loose)	0.03 to 0.05	0.09 to 0.12

(2) Elastic Method. For the elastic method of calculating single pile settlement, the designer is referred to Item 48.

(3) t-z Curve Methods. The t-z curve methods of calculating settlement of a single pile requires the use of a computer program and t-z curves (load transfer relationships for the pile-soil system). A number of computer programs are available from WES (Items 4, 13) for performing t-z curve analyses. Various load-transfer relationships (t-z curves) exist in supplemental literature (Items 9, 28, 29, 38, and 61).

b. Pile Groups. A number of methods exist for calculating settlement of groups of piles. The designer of a pile foundation should be aware that less is known about settlement of pile groups than any item discussed in this section.

(1) Group Settlement Factors. The simplest method for calculating settlement of a group of piles implements a group settlement factor.

$$S = \zeta_g w$$

where

S = settlement of a group of piles

ζ_g = group settlement factor

w = settlement of a single pile

The simplest expression for the group settlement factor is:

$$\zeta_g = \left(\frac{\bar{B}}{B}\right)^{0.5}$$

where

$\bar{B}$ = width of the pile group

B = diameter or width of a single pile

Three things must be kept in mind when using the above method:

(a) It is an approximate method.

(b) The group settlement factor was determined empirically from pile groups in sand.

(c) The settlement of a pile group is larger than that of a single pile with the same load per pile. This method takes that fact into account.

The following expression for the group settlement factor has been used for pile groups in clay:

$$\zeta_g = 1 + \sum_{i=1}^{n} \frac{B_i}{\pi S_i}$$

where

N = number of piles in group

s_i = distance from pile i to the location in the group where the group settlement is to be calculated

(2) Empirical Method. The empirical method for calculating the settlement of a group of piles is the method presented in Item 40. It is based on the concept that the pile group can be treated as an equivalent pier. For a group of friction piles, the equivalent footing is assumed to be founded at an effective depth of two-thirds of the pile embedment in the bearing stratum. For a group of end bearing piles, the equivalent footing is assumed to be founded at the pile tips.

(a) Groups in Sand. For calculating the settlement of pile groups in a homogeneous sand deposit not underlain by a more compressible soil at greater depth, the following expressions can be used:

$$S = \frac{2p\,\bar{B}}{\bar{N}} I$$

$$I = 1 - \frac{D'}{8\bar{B}} \geq 0.5$$

where

S = settlement of the pile group in inches

p = net foundation pressure, is defined as the applied load divided by the horizontal area of the group in tons per square foot

$\bar{B}$ = width of the pile group in feet

I = influence factor of effective group embedment

$\bar{N}$ = average corrected standard penetration resistance in blows per foot within the zone of settlement (extending to a depth equal to the pile group width beneath the pile tip in homogeneous soil)

D' = embedment depth of the equivalent pier

In using the above equation, the measured blow counts should be corrected to an effective overburden pressure of 1 ton per square foot as suggested in Item 46. The calculated value of settlement should be doubled for silty sand.

(b) Groups in Clay. The settlement of pile groups in clay or above a clay layer can be estimated from the initial deformation and consolidation properties of the clay. The pile group is treated as an equivalent pier and allowance is made for the effective foundation embedment and compressible stratum thickness as outlined above. (PHT reference)

(3) Elastic Methods. For the elastic methods of calculating settlements of a pile group, the designer is referred to Item 48. These methods require the estimation of a secant modulus.

(4) Stiffness Method. The stiffness method of analysis of pile groups as outlined in paragraph 4-5 can be used to calculate settlements of pile groups. As mentioned for other methods for calculating settlement of pile groups in clay, this method does not take into account any consolidation of the clay and must be corrected for settlement due to consolidation if such consolidation occurs.

4-5. Pile Group Analysis.

a. General. Several approximate methods for analysis of pile groups have been used. These graphical or numerical methods distribute applied loads to each pile within the group based on pile location, batter, and cross-sectional area. These approaches did not consider lateral soil resistance, pile stiffness, pile-head fixity, structure flexibility, or any effects of pile-soil interaction. Such factors significantly affect the distribution of forces among the piles and, if ignored, can result in an unconservative and erroneous pile design. Therefore, these methods should not be used except for very simple, two-dimensional (2-D) structures where the lateral loads are small (less than 20 percent of the vertical loads).

b. Stiffness Methods.

(1) General. The behavior of the structure-pile-soil system is non-linear. However, it is not practical to apply nonlinear theory to the

analysis and design of large pile groups in a production mode. Therefore, it is necessary to develop elastic constants which satisfactorily represent the nonlinear, nonelastic behavior. An approach for pile group analysis using force-displacement relationships has been developed. This method, referred to as the stiffness method, accounts for all of the variables mentioned above. The stiffness method is based on work published by A. Hrennikoff (Item 37). This method involves representation of individual pile-soil behavior by axial, lateral, rotational, and torsional stiffness constants. Individual pile forces are equal to the corresponding pile displacements times the pile-soil stiffness. Hrennikoff's analysis was restricted to two dimensions and piles with identical properties. Aschenbrenner (Item 26) extended the solution to three dimensions, and Saul (Item 54) used matrix methods to incorporate position and batter of piles, and piles of different sizes, and materials. The Saul approach is the basis for the pile analysis presented in the following paragraphs. These stiffness methods should be used for the analysis and design of all but the simplest pile groups. This method is implemented in the computer program CPGA (Item 5).

(2) Pile-Soil Model. In the stiffness method of pile analysis, the structure is supported by sets of six single degree-of-freedom springs which are attached to the base of the structure. These springs represent the action of the pile-soil foundation when the structure is displaced due to applied forces (Figure 4-7). The pile-load springs are linearly elastic and are used

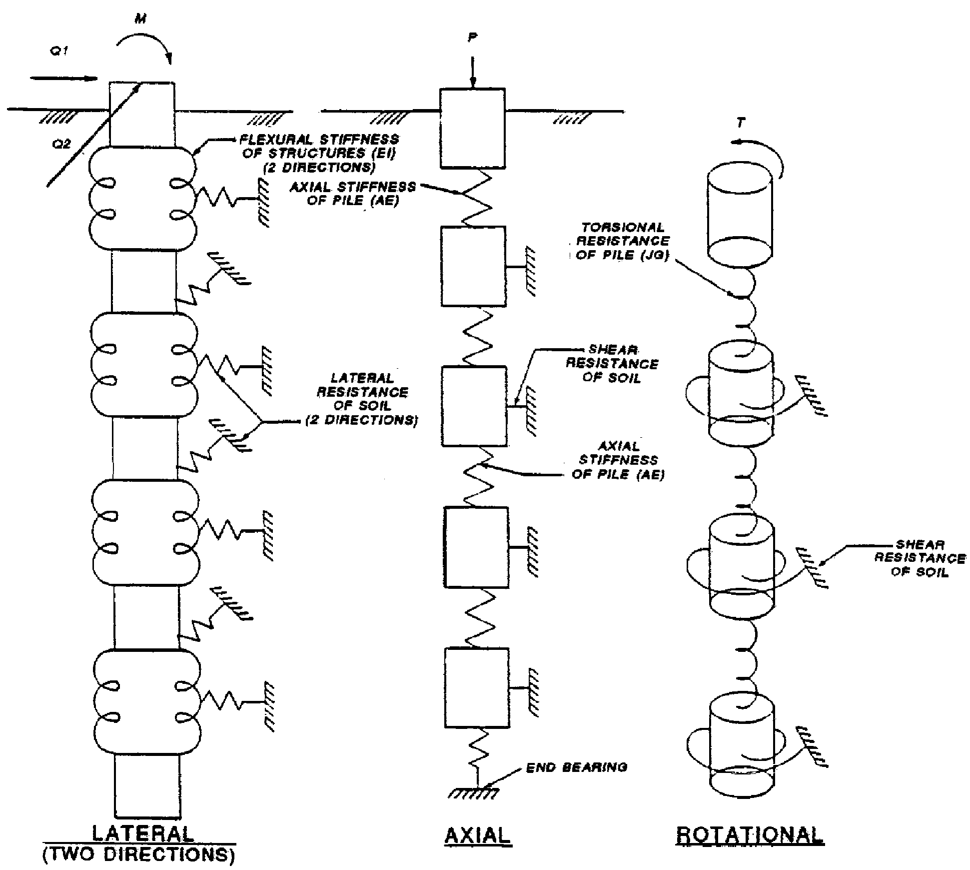

Figure 4-7. Spring model of pile-soil interaction

to account for all the variables and nonlinearities of the foundation. The behavior of each pile is represented by spring (or stiffness) constants in matrix form:

$$\{q\}_i = [B]_i \{u\}_i$$

where

$$(q)_i = \begin{Bmatrix} F_1 \\ F_2 \\ F_3 \\ M_1 \\ M_2 \\ M_3 \end{Bmatrix} = \text{pile forces and moments in } i^{th} \text{ pile}$$

$[B]_i$ = matrix of stiffness constants for i^{th} pile

$$(\mu)_i = \begin{Bmatrix} U \\ V \\ W \\ \Theta_1 \\ \theta_2 \\ \Theta_3 \end{Bmatrix} = \text{displacements and rotations of } i^{th} \text{ pile}$$

The total foundation stiffness is the summation of all the individual pile stiffnesses assembled into a global foundation stiffness matrix:

$$[K] = \sum_{i=1}^{n} [K]_i$$

where

$[K]$ = total pile group stiffness

n = number of piles in the foundation

$[K]_i$ = stiffness of i^{th} pile transformed to global coordinates

The elastic response of each pile to applied forces is based on a subgrade reaction assumption. This assumption is that the lateral resistance of the soil to pile displacements can be modeled as a series of linear springs connected to an individual pile. Therefore, the behavior of each set of springs is affected only by the properties of the pile and the surrounding soil and not by the behavior of adjacent piles. This approximation is necessary for computational simplicity and to allow for easy adaptability of the model to complications such as changes in soil type. Analytical results have been compared to actual field results from pile load tests for numerous cases and have demonstrated that this pile-soil model is satisfactory for the analysis of pile groups at working load. However, the designer should always be aware of the model limitations. A more realistic approach is being developed for design. Methods for determining the stiffness constants are presented in paragraph 4-5c.

(3) Rigid Base Versus Flexible Base. Distribution of the loads applied by the structure to each pile is affected by many factors. One important assumption is related to the flexibility of the pile cap. The pile cap (structure) can be modeled as a rigid or a flexible body. If the structure is assumed to behave as a rigid body, then the stiffness of the pile cap is infinite relative to the stiffness of the pile-soil system. For a rigid pile cap deformations within the structure are negligible, and the applied loads are distributed to each pile on the basis of rigid body behavior (Figure 4-8) as is the case in CPGA (Item 5). If the pile cap is assumed to be a flexible body, then the internal deformations of the structure are also modeled and play an important role in the distribution of the applied loads to each pile (Figure 4-9). When performing a pile group analysis, one of the first design decisions that must be made is how to model the flexibility of the structure. Parametric studies should be performed to determine the effects of the structure stiffness on the pile forces. For example, a pile-founded dam pier could be idealized using a 2-D beam element for the structure and springs for the piles. Available computer programs (such as SAP, STRUDL, CFRAME, etc.) can be used to vary the stiffness of the beams (structure), and the axial and lateral stiffness of the springs (piles), and thereby determine which pile cap assumption is appropriate. For either type of the pile cap, the piles are modeled as linear elastic springs.

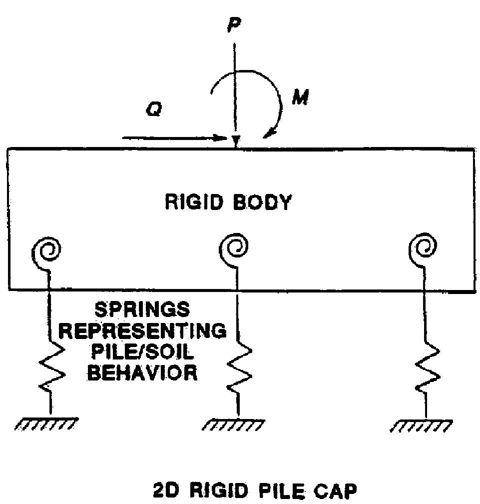

Figure 4-8. Rigid pile cap on a spring (pile) foundation

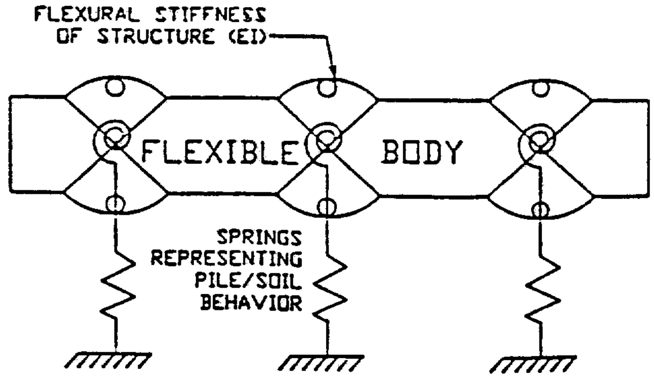

Figure 4-9. Flexible pile cap on spring (pile) foundation

(4) Nonlinear Effects. A pile group analysis is normally a linear elastic model. Actual load-deflection relationships for the pile-soil system can be nonlinear. Programs such as PILGP2R have been developed for nonlinear analysis of pile groups. The major disadvantage with using nonlinear pile group analysis programs is that they can only be used to analyze small pile groups, 30 piles or less. Many pile groups for hydraulic structures consist of 200 piles or more. Linear elastic pile group programs can approximate satisfactorily the nonlinear group analysis programs at working loads. A comparison was conducted for two typical pile groups using PILGP2R to perform a nonlinear analysis and using CPGA to perform a linear analysis which approximates nonlinear behavior (Item 45). The results for these two pile groups were in good agreement. The following methods for choosing stiffness coefficients are used to perform the linear CPGA analyses which approximately model nonlinear behavior.

c. Soil and Pile Properties. The soil-pile stiffness is a function of the pile structural properties, soil properties, degree of pile restraint against rotation, and pile-head movement. The pile properties needed to determine the spring stiffnesses are the modulus of elasticity, moment of inertia, cross-sectional area, width, and length. The soil properties needed to determine the spring stiffnesses are the undrained shear strength or angle of internal friction, and the unit weight. An estimate of pile-head movement is needed to determine the linear spring stiffnesses. This is accomplished by using a secant modulus corresponding to an estimated pile-head movement. If the calculated pile head movements reasonably agree with the estimated values, then the solution is acceptable; if not, then a new estimate of pile head movements must be used. (See paragraph 4-2e for additional discussion.)

d. Axial Stiffness. The axial pile stiffness is expressed as:

$$b_{33} = C_{33} \frac{AE}{L}$$

where

b_{33} = axial pile stiffness

C_{33} = constant which accounts for the interaction between the soil and the pile

A = cross-sectional area of the pile

E = modulus of elasticity of the pile

L = length of the pile

The term AE/L is the elastic stiffness of the pile acting as a short column with no soil present. The coefficient (C_{33}) accounts for the stiffness of the soil-pile system. The relationship between axial load capacity, movements of the pile head and tip, and load transfer along the shaft of friction piles is presented in the companion volume "Theoretical Manual for the Design of Pile Foundations," which is currently in preparation and is discussed in paragraph 1-3c(10).

(1) For design purposes, C_{33} for a compression pile ranges between 1.0 and 2.0 although values as low as 0.1 and as high as 3.0 have been noted in the literature. There appears to be a relationship between C_{33} and pile length. Longer piles tend to have higher values of C_{33} than shorter piles. C_{33} for tension piles in sand can be taken as one half of the value used for compression piles. For tension piles in clay use 75 to 80 percent of the value of C_{33} for compression piles.

(2) Long-term loading, cyclic loading, pile group effects, and pile batter can affect C_{33}. In sand, long-term loading has little effect on the value of C_{33}; however consolidation in clay due to long-term loading can reduce C_{33}. At present, the effect of cyclic loading on C_{33} is neglected. For design purposes, if piles are driven to refusal in sand or to a hard layer, there is no change in the value of C_{33} for pile groups; however, C_{33} may be reduced for groups of friction piles.

(3) The value of C_{33} for single piles can be calculated using the following equation:

$$C_{33} = \frac{\Delta}{\delta}$$

where

$\Delta = \dfrac{PL}{AE}$

δ = axial movement of the pile head due to axial load P

P = allowable axial design load for the pile

For axial stiffness, the load-deflection curve is essentially linear to one-half of the ultimate pile capacity (the design load), so nonlinearity of the axial pile stiffness can be neglected. Methods for calculating C_{33} from the above equations include empirical methods (Item 60), Winkler foundation analysis (Item 55), t-z curve analyses (Items 9, 28, 29, 38, and 61), finite element methods, and elastic method (Item 48). Values of C_{33} can be determined most accurately from pile load tests, where C_{33} can be determined to approximate the linear portion of the pile load-deflection curve.

e. Lateral Stiffness. Expressions for lateral pile stiffness are given in Item 17. The lateral pile stiffness expressions contain the following terms:

E = modulus of elasticity of the pile material

I = moment of inertia of the pile section

C_l = pile head-cap fixity constant (rotational restraint between pile head and pile cap)

E_s = modulus of horizontal subgrade reaction (expressed as soil reaction per unit length of pile per unit of lateral deflection)

n_h = constant of horizontal subgrade reaction (linear variation of E_s with depth i.e., $E_s = n_h x$)

Lateral pile stiffness expressions containing E_s (modulus of horizontal subgrade reaction not a function of depth) are assumed constant for overconsolidated clays. Lateral pile stiffness expressions containing n_h (modulus of horizontal subgrade reaction increasing linearly with depth) are used for sands and normally consolidated clays. Since the upper portion (10 pile diameters or less) of the soil profile usually controls the behavior of laterally loaded piles, most onshore clay deposits can be represented with a constant modulus of horizontal subgrade reaction. E_s and n_h are not constants. They both vary with deflection of the pile head. This is due to the fact that linear lateral stiffnesses are used to represent a nonlinear problem. To determine appropriate values of E_s or n_h, an estimate of lateral deflection must be made. If the calculated values of lateral deflection match the estimated values, then the correct value of E_s or n_h was used in the analysis. If not, a new value of E_s or n_h must be used based on the calculated deflection. For design, ranges of E_s or n_h are used to take into account variation of pile properties in different directions, variation of lateral pile deflection caused by different loading conditions, and variation of soil properties. After the analyses are completed, the calculated lateral deflection should be checked to make sure they correspond to the range of values of E_s or n_h assumed. If they do not, then the assumed range should be modified.

(1) Calculation of E_s or n_h. The first step in determining the range of E_s or n_h values to use in pile design is to determine curves of the variation of E_s or n_h with lateral pile head deflection. These curves can be estimated from plots of pile-head deflection versus applied lateral load (load-deflection curves). The pile-head, load-deflection curves can be obtained from lateral pile load tests or from p-y curve analyses (Items 13,

39, 50, 51, 52, and 53). From the load-deflection curves, the variation of E_s or n_h with deflection can be obtained using these equations for the case of applied groundline shear and zero applied moment.

$$n_h = \frac{C_n\left(\dfrac{P_t}{Y_t}\right)^{1.67}}{(EI)^{0.67}}$$

or

$$E_s = \frac{C_E\left(\dfrac{P_t}{Y_t}\right)^{1.33}}{(EI)^{0.33}}$$

where

C_n = 0.89 for a fixed-head pile or 4.41 for a free-head pile

P_t = lateral load applied at the top of the pile at the ground surface

Y_t = lateral deflection of the top of the pile at the ground surface

C_E = 0.63 for a fixed-head pile or 1.59 for a free-head pile

Use consistent units C_n and C_E are nondimensional constants.

(2) Stiffness Reduction Factors. Values of n_h or E_s calculated as outlined in the preceding paragraphs are for a single pile subject to static loading. Groups of piles, cyclic loading, and earthquake loading cause a reduction in E_s and n_h. Reducing E_s and n_h increases the pile deflections and moments at the same load level. The value of E_s or n_h for a single pile is divided by a reduction factor (R) to get the value of E_s or n_h for groups of piles, cyclic loading, or earthquake loading.

(a) Group Effects. Laterally loaded groups of piles deflect more than a single pile loaded with the same lateral load per pile as the group. This increased deflection is due to overlapping zones of stress of the individual piles in the group. The overlapping of stressed zones results in an apparent reduction in soil stiffness. For design, these group effects are taken into account by reducing the values of E_s or n_h by a group reduction factor (R_g). The group reduction factor is a function of the pile width (B), pile spacing, and number of piles in the group. Pile groups with center-line-to-center-line pile spacing of 2.5B perpendicular to the direction of loading and 8.0B in the direction of loading have no reduction in E_s or n_h. The group reduction factors for pile groups spaced closer than mentioned above are:

Center-Line-to-Center-Line Pile Spacing in Direction of Loading	Group Reduction Factor R_g
3B	3.0
4B	2.6
5B	2.2
6B	1.8
7B	1.4
8B	1.0

More recent data from pile group tests (Item 1, 8, and 12) suggest that these values are conservative for service loads, but at the present time no new procedure has been formalized.

(b) Cyclic Loading Effects. Cyclic loading of pile foundations may be due to tide, waves, or fluctuations in pool. Cyclic loading causes the deflection and moments of a single pile or a group of piles to increase rapidly with the number of cycles of load applied up to approximately 100 cycles, after which the deflection and moments increase much more slowly with increasing numbers of cycles. In design, cyclic loading is taken into account by reducing the values of E_s or n_h by the cyclic loading reduction factor (R_c). A cyclic loading reduction factor of 3.0 is appropriate for preliminary design.

(c) Combined Effect, Group and Cyclic Loading. When designing for cyclic loading of a group of piles, E_s or n_h for a single, statically loaded pile is divided by the product of R_g and R_c .

(d) Earthquake Loading Effects. The loading on the foundation induced by a potential earthquake must be considered in seismic active areas. The designer should first consider probability of an earthquake occurring during the life of the structure. If there is a likelihood of an earthquake occurring during the life of the structure, in seismic Zones 0 and 1 (EM 1110-2-1902), no reduction of E_s or n_h is made for cyclic loading due to short-term nature of the loading. In seismic Zones 2, 3, and 4, the potential liquefaction should be evaluated. If soils in the foundation or surrounding area are subject to liquefaction, the removal or densifaction of the liquefiable material will be necessary. Once the designer is assured that the foundation material will not liquefy, the analysis should be performed by Saul’s approach (Item 54) extended for seismic analysis as implemented in the computer program CPGD (Item 15).

(3) Stiffness Reduction Factor Equations.

(a) E_s-type Soil. For soils with a constant modulus of horizontal subgrade reaction, the following equations apply:

$$E_{s_{group}} = \frac{E_s}{R_g}$$

$$E_{s_{cyclic}} = \frac{E_s}{R_c}$$

$$E_{s_{group\ and\ cyclic}} = \frac{E_s}{(R_g R_c)}$$

$$Y_{t_{group}} = Y_t \ R_g^{0.75}$$

$$Y_{t_{cyclic}} = Y_t \ R_c^{0.75}$$

$$Y_{t_{group\ and\ cyclic}} = Y_t \ R_g^{0.75} \ R_c^{0.75}$$

(b) n_h-type Soil. For soils with a linearly increasing modulus of horizontal subgrade reaction, the following equations apply:

$$n_{h_{group}} = \frac{n_h}{R_g}$$

$$n_{h_{cyclic}} = \frac{n_h}{R_c}$$

$$n_{h_{group\ and\ cyclic}} = \frac{n_h}{(R_g R_c)}$$

$$Y_{t_{group}} = Y_t \ R_g^{0.6}$$

$$Y_{t_{cyclic}} = Y_t \ R_c^{0.6}$$

$$Y_{t_{group\ and\ cyclic}} = Y_t \ R_g^{0.6} \ R_c^{0.6}$$

(c) Definitions. The terms used in the above equations are:

$E_{s_{group}}$ = modulus of horizontal subgrade reaction for a pile in a pile group with static loading

E_s = modulus of horizontal subgrade reaction for a single pile with static loading

R_g = group reduction factor

$E_{s_{cyclic}}$ = modulus of horizontal subgrade reaction for a single pile with cyclic loading

R_c = cyclic loading reduction factor

$E_{s_{group\ and\ cyclic}}$ = modulus of horizontal subgrade reaction for a pile in a pile group with cyclic loading

$Y_{t_{group}}$ = pile head horizontal deflection at the ground surface for a pile in a pile group with static loading

Y_t = pile head horizontal deflection at the ground surface for a single pile with static loading

$Y_{t_{cyclic}}$ = pile head horizontal deflection at the ground surface for a single pile with cyclic loading

$Y_{t_{group\ and\ cyclic}}$ = pile head horizontal deflection at the ground surface for a pile in a pile group with cyclic loading

$n_{h_{group}}$ = constant of horizontal subgrade reaction for a pile in a pile group with static loading

n_h = constant of horizontal subgrade reaction for a single pile with static loading

$n_{h_{cyclic}}$ = constant of horizontal subgrade reaction for a single pile with cyclic loading

$n_{h_{group\ and\ cyclic}}$ = constant of horizontal subgrade reaction for a pile in a pile group with cyclic loading

(4) Pile Length. All of the lateral pile stiffness terms are based on the assumption that the piles are long and flexible as opposed to short and rigid. Piles are considered long if the applied lateral load at the head has no significant effect on the tip (the tip does not rotate or translate). Short piles behave rigidly and exhibit relatively no curvature (the tip rotates and translates). The computer programs referenced in this manual for group pile design are not intended for design of foundations containing short piles. Most piles used in the design of civil works structures are classified as long piles. The determination of the behavior of a pile as long or short is:

(a) Constant Modulus of Horizontal Subgrade Reaction.

$$\mathbf{R} = \sqrt[4]{\frac{EI}{E_s}}$$

$L/R \leq 2.0$; Short pile

$2.0 < L/R < 4.0$ Intermediate

$L/R \geq 4.0$ Long pile

(b) Linearly Increasing Modulus of Horizontal Subgrade Reaction.

$$T = \sqrt[5]{\frac{EI}{n_h}}$$

$L/R \leq 2.0$ Short pile

$2.0 < L/R < 4.0$ Intermediate

$L/R \geq 4.0$ Long pile

f. Torsional Stiffness. The torsional pile stiffness is expressed as:

$$b_{66} = C_{66} \frac{JG}{L}$$

where

b_{66} = torsional pile stiffness

C_{66} = constant which accounts for the interaction between the soil and the pile

J = polar moment of inertia of the pile

G = shear modulus of the pile.

L = length of the pile

The torsional stiffness of individual piles contributes little to the stiffness of a pile group for rigid pile caps and has been neglected in the past. More recent research has shown that a reasonable torsional stiffness is to use C_{66} equal to two. The coefficient C_{66} is equal to zero if the pile head is not fixed into the pile cap. See Items 44, 55, and 57 for details.

4-6. Design Procedure.

a. General. The following paragraphs outline a step-by-step procedure to design an economical pile foundation. The steps range from selection of applicable loads and design criteria through use of rigid and flexible base analyses. Identification and evaluation of foundation alternatives, including selection of the type of pile, are presented in Chapter 2.

b. Selection of Pile-Soil Model. A computer model (CPGS) is currently being developed and its capabilities are discussed in paragraph 1-3c(3), for analyzing the nonlinear interaction of the pile and surrounding soil. This model represents the lateral and axial behavior of a single pile under loading and accounts for layered soil, water table, skin friction, end bearing, and group effects. This computer model will be presented in detail in Mode CPGS. For large pile groups, the pile response is approximated by linear elastic springs. These springs represent the six degrees of freedom at the pile head, and their stiffnesses should be determined in close coordination between structural and geotechnical engineers. The designer should select a linear, elastic pile stiffness value for the group analysis by assuming a limiting deflection at the pile head. Then a secant pile stiffness should be determined for the assumed deflection using the nonlinear model or data from load tests conducted at the site. Deformations computed in the pile group analysis should be limited to this assumed deflection. The forces computed in the pile group analysis, using the secant pile stiffnesses, should be less than the actual forces from a nonlinear analysis (Figure 4-10). If more than 10 percent of the piles exceed the limiting deflection, a new secant pile stiffness should be developed for a larger limiting deflection. This method should be used in conjunction with interpretations of full-scale pile tests done at other sites that closely relate to the site under analysis. If site conditions are such that the foundation properties are not well defined, then a parametric approach should be used. A parametric analysis is performed by using stiff and weak values for the elastic springs based on predicted limits of pile group deflections. This parametric analysis should be applied to the lateral and the axial stiffnesses. See paragraph 4-5e for further discussion.

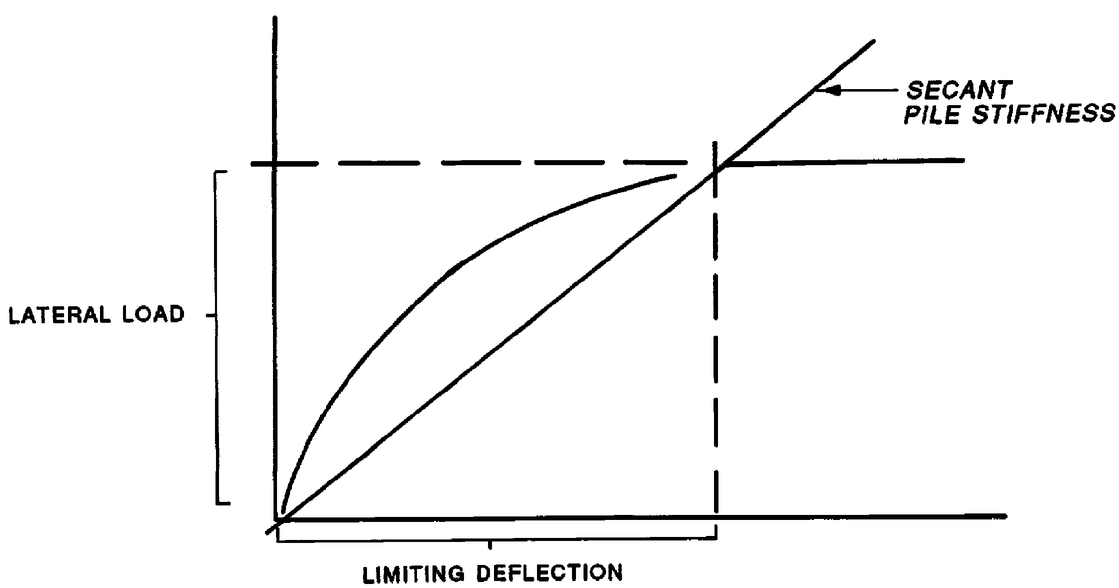

Figure 4-10. Pile forces for linear and nonlinear analysis

c. Selection of Pile Structure Model. The selection of the pile-structure model for analysis and design of a pile-founded structure must consider the following three critical items:

Type of structure (concrete or steel)

Type of analysis (rigid or flexible base)

Pile-head fixity (fixed, pinned, or partially fixed)

A reinforced concrete structure will require a rigid or flexible base analysis with the pile heads fixed or pinned. The decision regarding which type of base analysis to use is determined from the parametric analysis discussed in the preceding paragraph. A rigid base analysis should use the program CPGA (Item 5). A flexible base analysis should use one of the general purpose finite element computer programs, such as STRUDL or SAP, which have a pile element similar to the one used in CPGA. The flexible base analysis should be capable of handling all degrees of freedom for the two- or three-dimensional models. For example, to analyze a pile group with loading in the x, y, and z directions, the base should be modeled using plate elements or three-dimensional elements. For structures with loads in two directions only, a typical base strip should be modeled using frame elements as shown in Figure 4-11. Pile forces and moments and structure forces and moments are obtained from these analyses. An analysis of a steel frame on a pile foundation is accomplished in a similar manner. The degree of fixity of the pile to the steel frame must be included in developing the pile stiffnesses. The steel frame should be modeled as a space frame or plane frame supported by linear elastic springs which account for the degree of pile-head fixity. Pile forces and moments and frame forces and moments are obtained from this analysis. Earthquake loading in seismic areas must be considered. The program CPGD (Item 15) extends the three-dimensional rigid pile cap analysis of CPGA (Item 5) to provide a simplified, yet realistic, approach for seismic analysis of pile foundations. The CPGD program includes viscous damping of the pile-soil system and response spectrum loading. The CPGD program should be used during the seismic design process. Pile forces and moments are obtained from this seismic analysis.

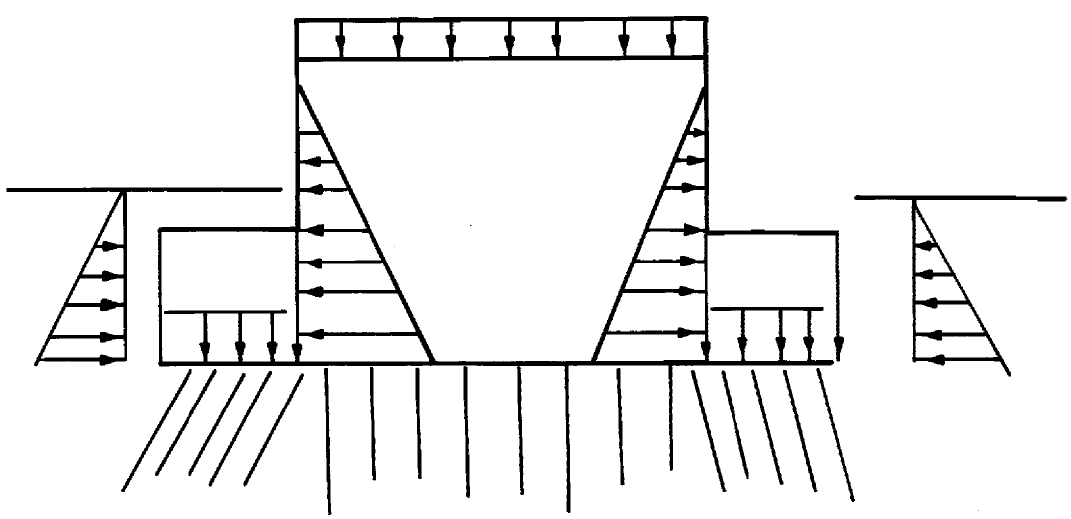

Figure 4-11. Typical 2-dimensional base strip modeled using frame elements

d. Selection of Load Cases. General loading conditions should be identified, and each condition should be assigned an appropriate safety factor and allowable stress. Study of the list of loading cases will reveal that some load cases will not control the design and should be eliminated. The remaining load cases should be studied in more detail. Loading details should be established to produce critical combinations. Consider the effect each load will have on pile forces and on internal forces in the pile cap. Some loadings may control the internal design of the pile cap even though they may not produce the critical pile forces. Generally, it is important to analyze the load cases with the largest lateral loads in each direction and the cases with the maximum and minimum vertical loads. Final selection of the load cases should be based on engineering judgement.

e. Selection of Design Criteria. Paragraph 4-2 provides specific guidance about safety factors, pile stresses, and pile cap movements. Criteria for ultimate pile capacity are presented in paragraph 4-3, and development of pile stiffness values is described in paragraph 4-5. These criteria may be applied to most pile foundation designs. However, uncertainty about pile-soil behavior may require modification of some criteria to ensure a conservative design. The magnitude of the lateral or axial pile stiffness may significantly affect the results of any pile analysis. Combining limiting values of lateral and axial pile stiffnesses may result in significantly different percentages of the applied loads being resisted by pile bending or axial force. This is particularly important for flexible base analyses because the applied loads are distributed to the piles based on the relative stiffness of the structure and the piles. Therefore realistic variations in pile stiffnesses should usually be evaluated, and the pile group should be designed for the critical condition. The variation of stiffnesses should correspond to the predicted deflection of the pile group.

f. Deformations. The pile stiffnesses in the lateral and axial directions is determined by a nonlinear analysis assuming a limiting deformation. Since the pile stiffness is a secant model of the pile response, the calculated deflections of the pile head under working loads should be limited to that assumed value. If the analysis yields deformations greater than those assumed in determining the pile stiffnesses, then the geotechnical engineer should be consulted and the stiffnesses should be reevaluated. Calculated pile cap deformations should be checked against functional and geometric constraints on the structure. These values are usually 1/4-inch axially and 1/2-inch laterally. For unusual or extreme loads these values should be increased.

g. Initial Layout. The simplest pile layout is one without batter piles. Such a layout should be used if the magnitude of lateral forces is small. Since all piles do not carry an equal portion of the load, axial pile capacity can be reduced to 70 percent of the computed value to provide a good starting point to determine an initial layout. In this case, the designer begins by dividing the largest vertical load on the structure by the reduced pile capacity to obtain the approximate number of piles. If there are large applied lateral forces, then batter piles are usually required. Piles with flat batters, 2.5 (V) to 1 (H), provide greater resistance to lateral loads and the less resistance to vertical loads. Piles with steep batters, 5 (V) to 1 (H), provide greater vertical resistance and less lateral resistance. The number of batter piles required to resist a given lateral load can also be estimated by assuming that the axial and lateral resistances are approximately 70 percent of computed capacity. This should be done for the steepest and flattest batters that are practical for the project, which will provide a range estimate of the number of batter piles required. For a single load case this method is not difficult. However, when the pile group is subjected to several loading conditions, some with lateral loads applied in different directions, this approach becomes more difficult. For such cases, two or three critical loading conditions should be selected to develop a preliminary layout from which the number, batters, and directions of piles are estimated. A uniform pile grid should be developed based on the estimated number of piles, the minimum pile spacing and the area of the pile cap. If piles with flat batters are located in areas of high vertical loads, then vertical piles should be placed adjacent to these battered piles. An ideal layout for flexible structures will match the pile distribution to the distribution of applied loads. This match will result in equal loads on all piles and will minimize the internal forces in the structure because the applied loads will be resisted by piles at the point of loading. For example, a U-frame lock monolith has heavy walls and a relatively thin base slab. Therefore, piles should be more closely spaced beneath the walls and located at larger spacings in the base slab.

h. Final Layout. After the preliminary layout has been developed the remaining load cases should be investigated and the pile layout revised to provide an efficient layout. The goal should be to produce a pile layout in which most piles are loaded as near to capacity as practical for the critical loading cases with tips located at the same elevation for the various pile groups within a given monolith. Adjustments to the initial layout by the addition, deletion, or relocation of piles within the layout grid system may be required. Generally, revisions to the pile batters will not be required because they were optimized during the initial pile layout. The designer is cautioned that the founding of piles at various elevations or in different strata may result in monolith instability and differential settlement.

i. Design of Pile Cap. If the pile group is analyzed with a flexible base, then the forces required to design the base are obtained directly from the structure model. If the pile group is analyzed with a rigid base, then a separate analysis is needed to determine the stresses in the pile cap. An appropriate finite element model (frame, plate and plane stress or plane strain) should be used and should include all external loads (water, concrete soil, etc.) and pile reactions. All loads should be applied as unfactored service loads. The load factors for reinforced concrete design should be applied to the resulting internal shears, moments, and thrusts acting at each cross section. The applied loads and the pile reactions should be in equilibrium. Appropriate fictitious supports may be required to provide numerical stability of some computer models. The reactions at these fictitious supports should be negligible.

4-7. Special Considerations.

a. Soil-Structure Interaction. Pile-supported structures should be analyzed based on the axial and lateral resistance of the piles alone. Additional axial or lateral resistance from contact between the base slab and the foundation material should be neglected for the following reasons. Scour of the riverbed frequently removes material from around the slab. Vibration of the structure typically causes densification of the foundation material and creates voids between the base slab and foundation material. Also, consolidation or piping of the foundation material can create voids beneath the structure.

b. Deep Seated Lateral Movement and Settlement. The soil mass surrounding a pile group must be stable without relying on the resistance of the pile foundation. In actual slides, 48-inch diameter piles have failed. Deep seated stability of the soil mass should be analyzed by neglecting the piles. Potential problems of inducing a deep seated failure due to excess pore water pressures generated during pile driving or liquefaction due to an earthquake should be recognized and accounted for in the design. The probable failure mechanism for piles penetrating a deep seated weak zone is due to formation of plastic hinges in the piles after experiencing large lateral displacements. Movement in the weak zone will induce bending in the piles as shown in Figure 4-12. A second mechanism is a shear failure of the piles which can only occur if the piles penetrate a very thin, weak zone which is confined by relatively rigid strata. The shear force on the piles can be estimated along the prescribed sliding surface shown in Figure 4-13. Research is being sponsored at the University of Texas which will develop a practical approach to solve these problems. The results of this research will be included in this manual and the capabilities of CPGA (Item 5) and CPGS, paragraph 1-3c(3), for analyzing such situations will be extended. Downdrag due to settlement of the adjacent soil mass may induce additional loads in the piles.

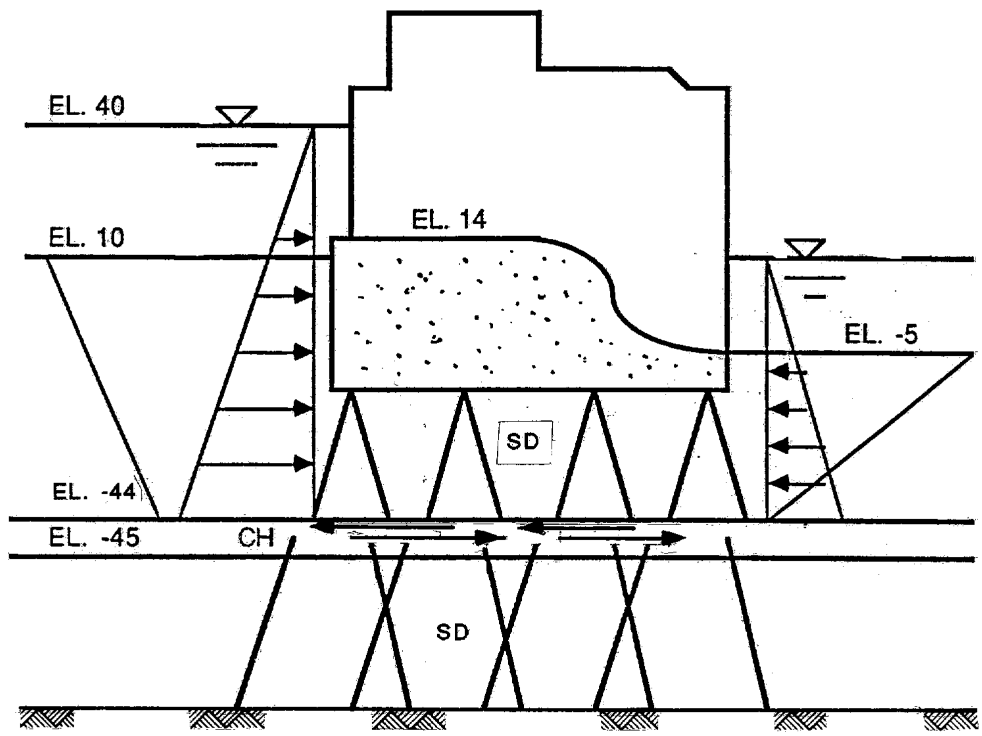

Figure 4-12. Piles are sheared off by the massive soil movement

c. Differential Loadings on Sheet-Pile Cutoffs. The length of a sheet pile cutoff should be determined from a flow net or other type of seepage analysis. The net pressure acting on the cutoff is the algebraic sum of the unbalanced earth and water pressures. The resulting shear and moment from the net pressure diagram should be applied to the structure. For flexible steel sheet piles the unbalanced load transferred to the structure may be negligi ble. For a continuous rigid cutoff, such as a concrete cutoff, the unbalanced load should be accounted for. An example is shown in Figure 4-14.

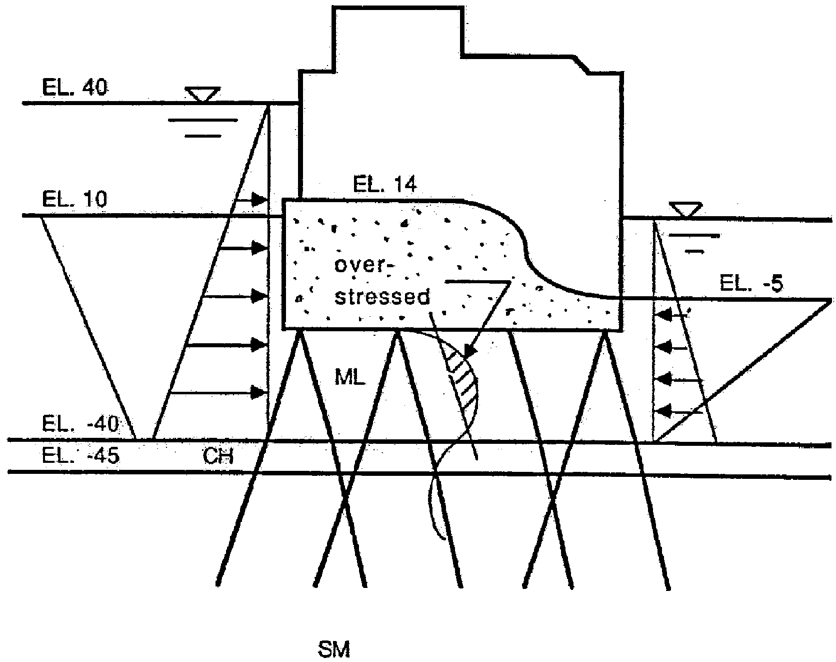

Figure 4-13. Piles are overstressed by bending moment

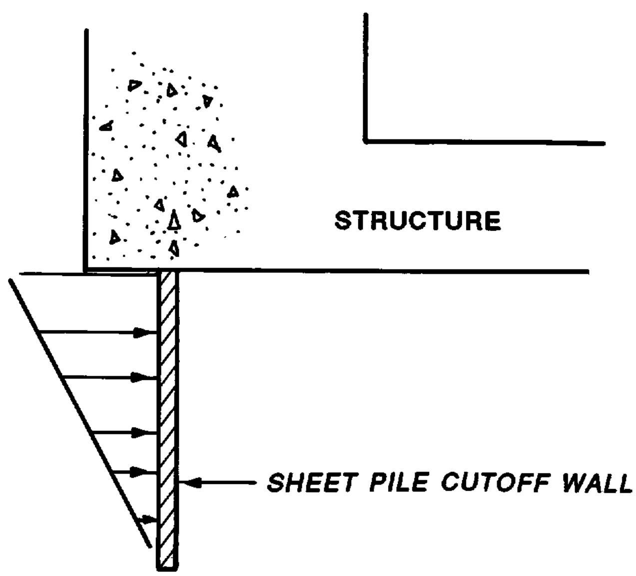

Figure 4-14. Pressure distribution on sheet pile cut off wall

d. Effects of Changes in the Pile Stiffness.

(1) General. Accurate predictions of the soil-pile stiffnesses for a specific site and set of construction circumstances are extremely difficult. The interaction of the structure-pile-soil system is complex and is usually nonlinear. The load deformation behavior of this system is affected to varying degrees by the type of loading, pile spacing, pile head fixity, subgrade modulus, pile-driving procedures, water table variations, and other variables. The designer should account for these uncertainties and variations by judiciously selecting a realistic range of pile stiffnesses, and by evaluating the sensitivity of the pile forces, moments and displacements to reasonable variations in the pile stiffnesses. This procedure should be used to develop a high degree of confidence in the design.

(2) Rigid Base. For a pile group that contains only vertical piles, the rigid cap assumption requires that the plane of the pile heads remains plane when loads are applied. Therefore, since the axial and lateral components of the pile reactions are independent, changes in the axial or lateral pile stiffnesses will have predictable results. If the pile layout contains a combination of vertical and batter piles, then the interaction of lateral and axial components of the pile reactions can have significant and often unforeseen effects on the distribution of pile forces. Therefore, changes in the lateral stiffnesses could have a profound effect on the axial pile forces, and the sensitivity of the pile forces to changes in the pile stiffnesses would not be predictable without using a computer analysis. See Item 3 for example.

(3) Flexible Base. When the stiffness of the structure is not infinite compared to the stiffness of the pile-soil system, the pile cap is assumed flexible. The sensitivity of the pile loads to changes in the pile stiffness then becomes even more difficult to predict. The axial and lateral response of the piles are interrelated, and the internal stiffness of the structure significantly influences the distribution of the individual pile loads. Changes in the pile stiffnesses can also affect the deformation characteristics of the structure, thereby changing the internal moments and member forces. Figure 4-15 illustrates the effects of changing the stiffness of pile cap. In Figure 4-16 the base of the infinitely rigid pile cap deflects uniformly, causing uniform loads in the piles and large bending moments in the base slab. If the slab stiffness is modeled more realistically, as shown in Figure 4-15, the pile loads will vary with the applied load distribution. The pile loads will be lower under the base slab causing the base slab moments to be reduced. The correct stiffness relationship between the structure and the foundation is extremely important for accurately designing a pile group.

(4) Confidence Limits. An essential element of all pile foundation designs is the effort required to define the stiffness of the structure-pile-soil system confidently. Initial pile stiffnesses should be selected and used to perform a preliminary analysis of critical load cases. If the preliminary analysis indicate that the selected pile stiffnesses are not sufficiently reliable, and that the variation of the pile stiffnesses will significantly affect the analytical results, then more intensive investigation is required. Normally a limit analysis is performed to bracket the solution. With this limit approach, all the factors which tend to minimize the pile-soil resistance are collectively used to represent a weak set of pile stiffnesses. This condition is a lower bound. Similarly, all the factors which tend to maximize

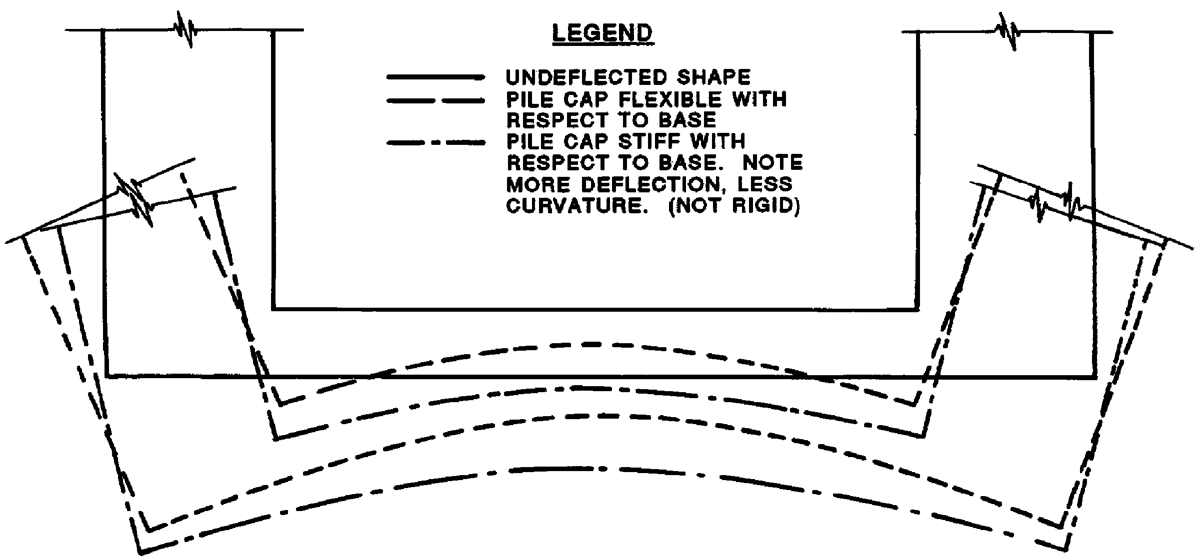

Figure 4-15. Deflected shape of flexible pile caps

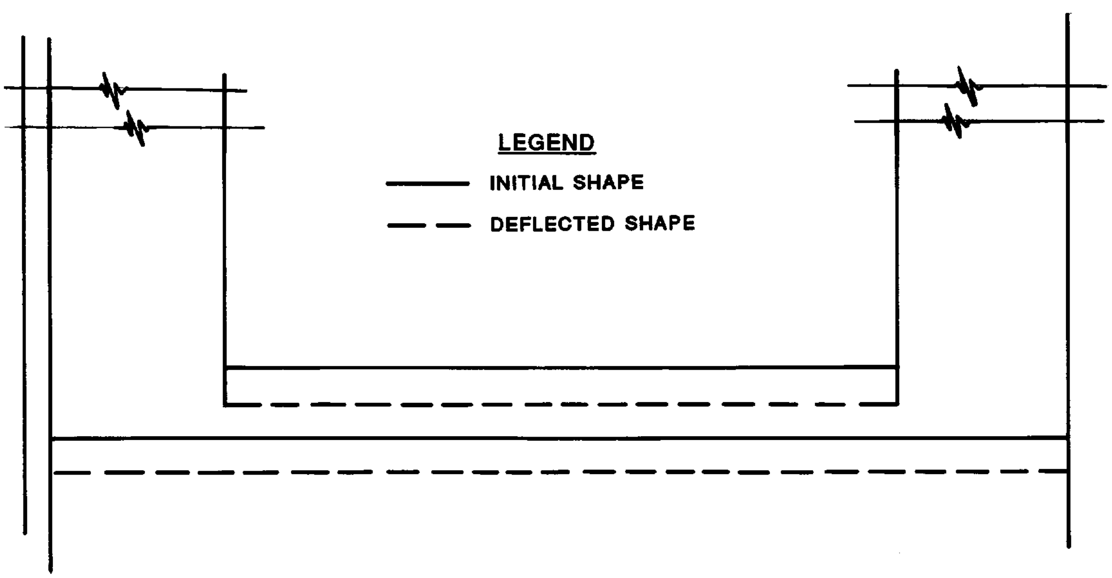

Figure 4-16. Deflected shape of a rigid pile cap

pile-soil resistance are collectively used to represent a strong set of pile stiffnesses (upper bound). Using this procedure, the designer can establish confidence limits by performing two analyses which bracket the actual set of parameters. For further discussion of this procedure, refer to Paragraph 4-6.

e. Effects of Adjacent Structures.

(1) General. Most hydraulic structures are designed to function as independent monoliths. Sometimes it is necessary to design hydraulic structures which interact with adjacent monoliths or existing structures. Certain procedures and details should be used to assure that the actual structural performance is consistent with the design assumptions.

(2) Independent Monoliths. Generally, hydraulic structures should be designed to function as independent monoliths. Each monolith should be isolated by vertical joints and should not interact with adjacent monoliths. This approach greatly simplifies the analysis and is a reliable basis for predicting performance. Validity of the design assumptions should be assured by including the following procedures and details. Independent monoliths should not be physically connected to adjacent monoliths. Expansion joints should be provided between monoliths to accommodate the predicted displacements. Rigid cap displacements should be extrapolated to the top of the monolith and the displaced structure should not make contact with adjacent monoliths. Batter piles should not interfere (based on common construction tolerances) with piles under adjacent structures. It is good design practice, but not always practical, to keep the tips of all piles within the perimeter of the pile cap. Possible interferences with piles under adjacent monoliths should be checked using CPGI, a program currently being developed and discussed in paragraph 1-3c(6), and the pile layout should be modified as needed.

(3) Interacting Monoliths. Sometimes it is necessary to design the pile groups of adjacent monoliths to interact and resist large unbalanced lateral loads. There are three types of circumstances:

(a) Analysis of new structures that are geometrically constrained from permitting sufficient batter and numbers of vertical piles to resist the lateral forces.

(b) Analysis of new structures that are subjected to a highly improbable loading condition. Such extreme lateral loads make it economically unfeasible to design a pile layout for independent adjacent monoliths.

(c) Evaluation of existing structures.

For designing new structures, provisions should be included to assure positive load transfer between monoliths (preferably at the pile cap) and without causing detrimental cracking or spalling. Provisions should be included for keying and grouting the monolith joint between the pile caps of interacting structures. Special attention should be given to the monolith joints in the thin wall stems of U-frame locks. The wall joints should be detailed to accommodate monolith movements without significant load transfer and thereby control localized cracking and spalling. For evaluating existing structures, the analyst should model actual field conditions as closely as practical. Field measurements should be made to determine pile-cap displacements and changes in monolith joint dimensions. Investigations should determine if load transfer is occurring between monoliths (joint closure, spalling concrete at joints, etc.). Foundation investigations should be adequate to estimate the lateral and axial pile stiffnesses.

(4) New Structures Adjacent to Existing Structures. Special provisions are appropriate for designing and installing piles adjacent to an existing structure. Existing structures include those under construction or already in service. During construction, pile driving should not be allowed within 100 feet of concrete which has not attained its design strength. Pile driving within 100 feet of concrete that has achieved the required design strength should be monitored for detrimental effects on the existing concrete. If piles are installed near an existing structure, it is prudent to monitor and document effects of pile driving on the existing structure and foundation.

Such provisions should be fully considered during design. Potential damage to existing structures may be influenced by a variety of factors:

(a) Densification of existing fill may induce settlement and a significant increase in lateral earth pressures.

(b) Driving displacement piles in noncompressible materials may cause heave of the ground surface.

(c) Driving piles in submerged, uniformly fine-grained, cohesionless soils may rearrange the soil grains and increase groundwater pressure with corresponding large settlements.

(d) Lateral load resistance of adjacent pile foundations may be significantly reduced.

These factors and others should be thoroughly investigated during design.

(5) Special Techniques. Special types of pile installation should be used to minimize possible damage. These may include:

(a) Using nondisplacement piles.

(b) Specifying a pile hammer that minimizes vibrations.

(c) Jacking piles.

(d) Using predrilled pilot holes or jetting.

The condition of existing structures and the surrounding area should be carefully documented before, during, and after pile driving. Field surveys, measurements, photographs, observations, sketches, etc. should be filed for future reference.

f. Overstressed Piles. The design criteria in preceding paragraphs are generally applicable for each load case. However, on large foundations, a few piles may exceed the allowable capacity or stresses by a relatively small amount without endangering the integrity of the structure. The design of a pile group should not be dictated by localized overload of a few corner piles for one load case. Because of the highly nonlinear load-deflection relationship of piles and the large plastic ranges that some piles exhibit, high localized pile loads are usually redistributed without danger of distress to adjacent piles until a stable state of equilibrium is attained. The stiffness method of analysis is an approximate linear model of the nonlinear load deflection behavior of each pile. Since the stiffness analysis is not exact a few piles may be loaded above the allowable capacity. Iterative pile group analyses are required.

g. Pile Buckling. Buckling of individual piles is related to the load level, the flexibility of the pile cap, the geometry of the pile group, and the properties of the soil and piles. Pile-soil stiffness and the degree of lateral support provided by the soil primarily depend on the following factors:

(1) Embedment. If the piles are fully embedded, then the lateral support provided by the soil is usually sufficient to prevent pile buckling. Even extremely weak soils may provide sufficient support to prevent buckling when fully embedded. Buckling may be critical if the piles project above the surface of soils that provide strong lateral support.

(2) Rigidity. The pile shape (radius of gyration), modulus of elasticity of the pile, the lateral and axial support provided by the soil, the degree of fixity of the pile head, and the flexibility of the pile cap all affect the relative pile rigidity. Buckling analysis is very complex because the axial and transverse loads and the pile stiffnesses affect the deformation of the pile, and this behavior is related through interaction with the soil.

(3) Tip Resistance.

h. Pile Splicing.

(1) General. The probability and reliability of splicing piles should be considered early in design. The structural integrity of the piles and complexity of the installation procedures must be comprehensively evaluated before selecting the location and types of splices allowed. Most splicing is performed in the field and significantly increases construction time, cost, and the field inspection required to assure reliability. Therefore, field splicing is normally limited to situations where only occasional splices are required. Splicing may be necessary in construction areas with limited overhead clearances or if the pile does not attain its required design capacity at the specified tip elevation. Contract plans and specifications should address the use (or exclusion) of splices and any specific requirements or limitations that are necessary. Splicing should not be allowed in the field without prior consent and approval of the designer.

(2) Structural Integrity. Splices should be capable of resisting all forces, stresses, and deformations associated with handling, driving, service loads, or other probable sources. Splices in the upper portion of the pile should be designed to account for the possible effects of accidental eccentric loadings. Regions of low bending and shear stresses under service loads are preferable for splice locations. Allowable stresses should be limited to those listed in paragraph 4-2d, and deformations should be compatible with the interaction between the pile and structure. The design should also account for the effects of corrosion and cyclic or reverse loading if present. Many commercial splices are not capable of developing the full strength of the pile in tension, shear, and bending.

(3) Soil Integrity. Splice surfaces which extend beyond the perimeter of the pile may disturb the interface between the pile and soil during driving and decrease adhesion. If appropriate, reductions in axial and lateral pile capacities should be made. This condition is most likely to occur in stiff clays, shales, and permafrost.

(4) Installation. Most splicing is performed in the field, sometimes in the driving leads. Engineering experience and judgement are essential in assessing the critical factors influencing reliability and cost (i.e. field access to the splice location, workmanship and quality assurance). The time required to perform the splice is also critical if the pile tends to set and become more difficult to restart when driving resumes. Piles driven into

materials with high adhesion or granular materials exhibit rapid set to a greater degree than soft clays or sensitive soils.

i. As-Built Analyses. As explained in paragraph 5-6a, conditions encountered in the field may result in variations between the pile foundation design and the actual pile foundation. All such variations should be observed, recorded and evaluated by the designer in an as-built analysis. The number of overloaded piles, the severity of the overload, and the consequences of the failing of one or more overloaded piles should be evaluated in the as-built analysis. Structural deformations and interaction between adjacent monoliths also could be significant factors.

(1) Geometric Factors. Field conditions may cause variations in the geometric layout of individual piles; i.e., pile head may move horizontally or rotate, batter may change, and final tip elevation may vary due to a change in batter or soil properties. Such geometric variations may substantially affect the individual pile loads even though the pile capacity remains unchanged.

(2) Soil Properties. Variations in soil properties may affect pile-tip elevations, pile capacities and the axial and lateral pile stiffnesses.

(3) Obstructions. Unexpected subsurface obstructions may prevent driving some piles to the design tip elevation, thereby causing variations in the pile stiffnesses or necessitating field changes.